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STUDENT WORKBOOK

SOILS

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US ARMY ENGINEER SCHOOL - FORT BELVOIR, VIRGINIA

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INSTRUCTOR'S NAME____________________ROOM______PHONE____________

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REFERENCES: TM 5-330 (SEP68) PLANNING AND DESIGN OF ROADS, AIRBASES, AND HELIPORTS IN THE THEATER OF OPERATIONS; para.2-1 through 2-33, para.2-86 through 2-93, and Chapter 9.

TM 5-530 (FEB71) MATERIAL TESTING, Chapters 2 and 5

FM 5-34 (SEP76) ENGINEER FIELD DATA, Chapter 9

TM 5-545 (JUL71) GEOLOGY

BASIC ROCK PROPERTIES AND SITE LOCATION

I. Engineering Properties of Rocks

A. The objective of this section is to familiarize the student with basic properties of rocks and where each type is likely to be found.

B. Field evaluation of rocks and their properties.

1. Toughness - is the mechanical strength of the rock, its resistance to being crushed or broken.

   Field test - Strike the rock with a hammer and listen to the sound it makes. A ringing sound indicates a tough rock while a thud means it is a weak rock.

   Suitability for construction - The tougher the rock the better it is for construction.

2. Hardness - is the resistance to being scratched or worn.

   Field test - Scratch rock against an object of known hardness. If the rock scratches the object the rock is harder; if the rock is scratched by the object the rock is softer. Compare rock with several objects until approximate hardness is determined. See Table 1-1 for some expedient objects.

   Suitability for construction - Rocks harder than steel can cause excessive damage to crushers. Very soft rocks are not durable enough.

Table 1-1 Expedients

Material	Hardness
Porcelain	7.0
Steel file	6.5
Window glass	5.5
Knife blade or nail	5.0
Copper coin (penny)	3.0
Fingernail	2.0

NOTE: Scale is from 1 to 10. 1 is the softest and 10 is the hardest rock known.

3. Density - is the weight per unit of volume.

   Field test - Heft two rocks of equal size; the heavier rock is more dense.

   Suitability for construction - Denser rocks are more resistant to erosion, but are more difficult and expensive to excavate and haul.

4. Durability - is the resistance to weathering.

   Field test - Look at naturally exposed rock surfaces to see the effects of weathering on that type of rock.

   Suitability for construction - more durable rock is better, but durability is not as critical as other properties for T/O construction.

5. Crushed shape - is the shape of broken fragments of a larger rock.

   Field test - Shatter a rock sample with a hammer and observe the broken pieces of the rock.

   Suitability for construction - Angular, bulky fragments make better aggregate than do platy, stringy, or rounded fragments.

6. Surface character - is the roughness of a broken rock surface which determines the bonding characteristics for that rock.

   Field test - Feel a piece of the rock with your finger or thumb.

   Suitability for construction - A rough but not excessively jagged surface is best; not smooth but not too sharp.

7. Chemical stability - is the resistance to the alkalies in portland cement.

   Field test - Break rock with a hammer. If the fresh surface is glassy, and the rock breaks into slivers or fragments like glass, the rock could be obsidian or chert. However, there is no test that will detect all unsuitable rocks.

   Suitability for construction - Obsidian, chert, and some other rocks react with the alkalies in portland cement and cause expansion, cracking, and deterioration in concrete.

II. Means of Aggregate Location

A. Reference information sources

1. Strategic Intelligence
2. Geologic Maps
3. Topographic Maps
4. Agricultural Soils Maps
5. Aerial Photographs
6. Foreign Maps and Publications
7. Tactical Intelligence
8. Local Inhabitants
9. Miscellaneous Records

B. Map reconnaissance

1. Is used to find probable locations of construction materials.

2. Stream deposits - SEE FIGURES 1-1 through 1-7

    a. Point bars - Where coarse-grained sediments are deposited on the inside of a bend.

    ________ and _______ are usually found here.

    b. Channel bars - are coarse deposits just downstream and on the opposite side of the stream from a point bar.

    ________ and _______ are often found here.

FIGURE 1-1. STREAM DEPOSITS

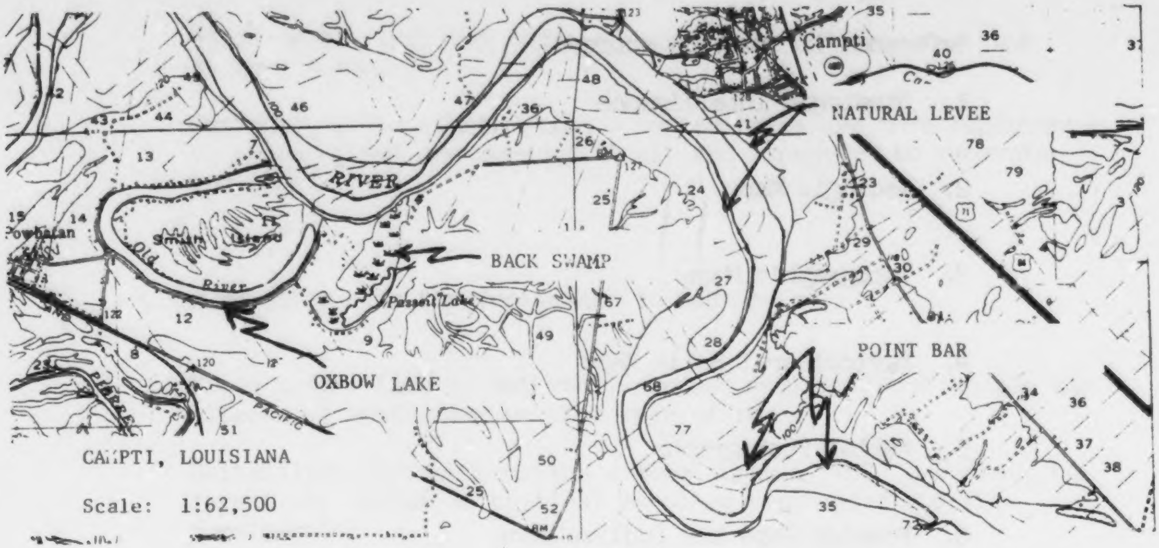
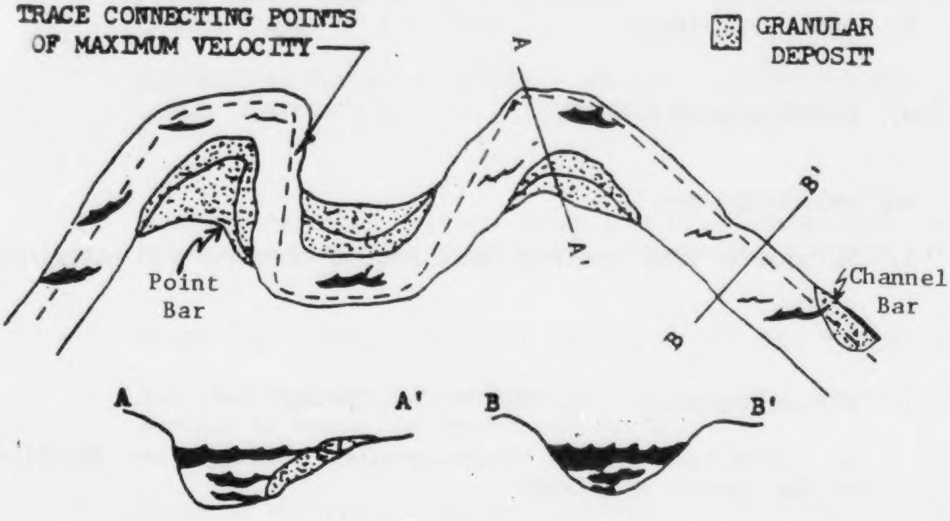

Campti
NATURAL LEVEE
RIVER
Smith Bend
Old River
BACK SWAMP
Powhatan
Pusson Lake
POINT BAR
OXBOW LAKE
CAMPTI, LOUISIANA
Scale: 1:62,500

FIGURE 1-2. DEPOSITS ALONG MEANDERING STREAMS


TRACE CONNECTING POINTS OF MAXIMUM VELOCITY
▨ GRANULAR DEPOSIT
A
A'
B
B'
Point Bar
Channel Bar
A
A'
B
B'

c. Oxbow lakes - are where a bend has been cut off by continuing stream activity.

________ and ________ are found here.

d. Natural Levees - are deposits of material along the edges of a stream.
NOTE: THESE LEVEES ARE NOT ADEQUATE FOR FLOOD CONTROL.

________ is usually found here.

Characteristics: Good for bridge foundations
Good trafficability on crest of levee, but
POOR trafficability on other parts

e. Backswamps - are the area in the floodplain between the natural levees and alluvial terraces.

____________ is usually found here.

Characteristics: POOR trafficability

f. Alluvial terraces - are deposits left by the stream when it had a higher water level or was at a higher elevation.

_________ and _________ are found here.

Characteristic: EXCELLENT trafficability

FIGURE 1-3. ALLUVIAL VALLEY (CROSS-SECTION VIEW)

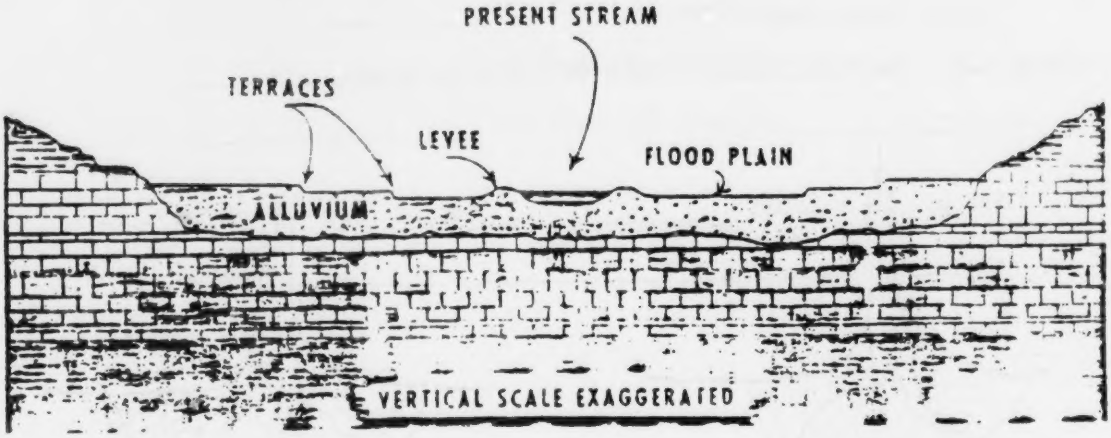
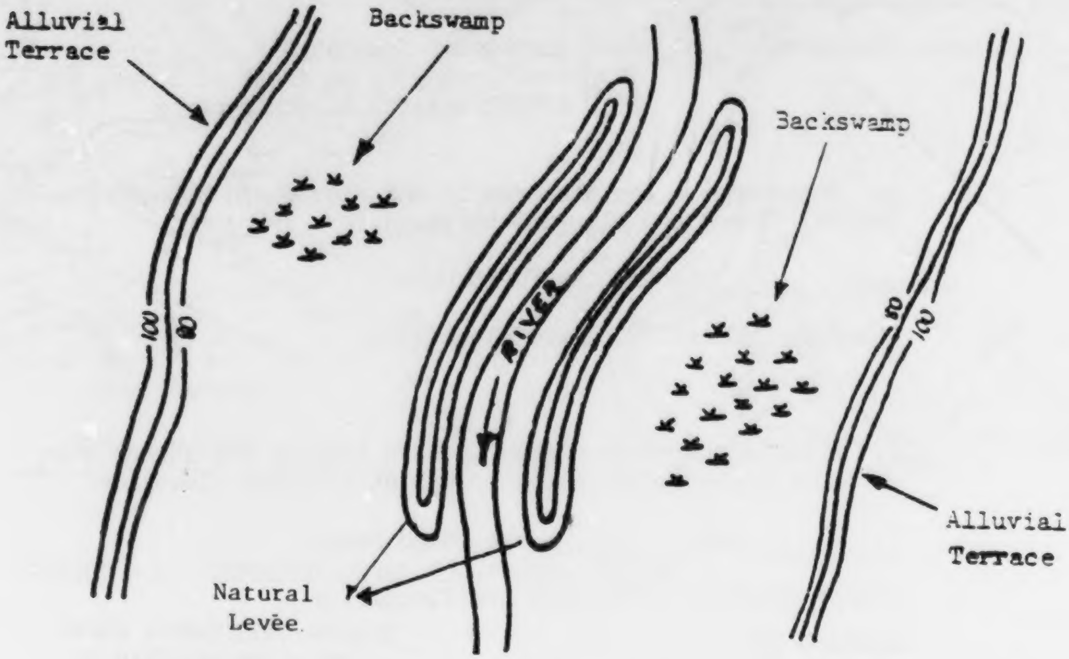
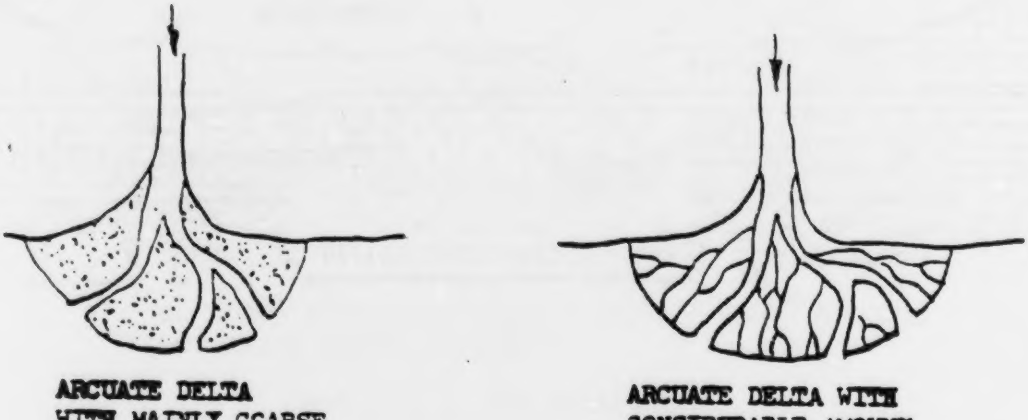

PRESENT STREAM
TERRACES
LEVEE
FLOOD PLAIN
ALLUVIUM
VERTICAL SCALE EXAGGERATED

FIGURE 1-4. ALLUVIAL VALLEY (MAP VIEW)


Alluvial Terrace
Backswamp
100
80
RIVER
Backswamp
80
100
Alluvial Terrace
Natural Levèe

g. Deltas - are deposits where a stream empties into a large body of water.

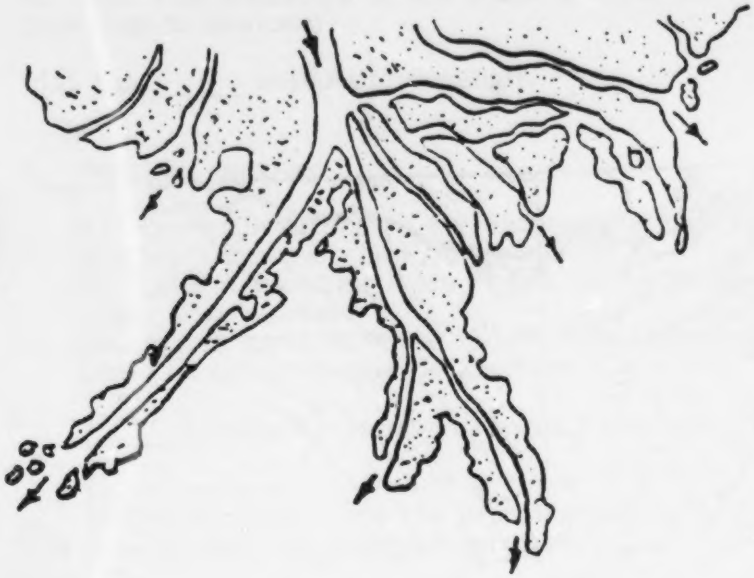
FIGURE 1-5. ARCUATE DELTAS (such as the Nile River)



ARCUATE DELTA WITH MAINLY COARSE MATERIAL

ARCUATE DELTA WITH CONSIDERABLE AMOUNT OF FINES WITH THE COARSE MATERIAL

FIGURE 1-6. BIRD'S-FOOT DELTA (example: the Mississippi River)



h. Alluvial fans - are formed by a stream flowing down a hillside.

____________________ are found in the __________________.

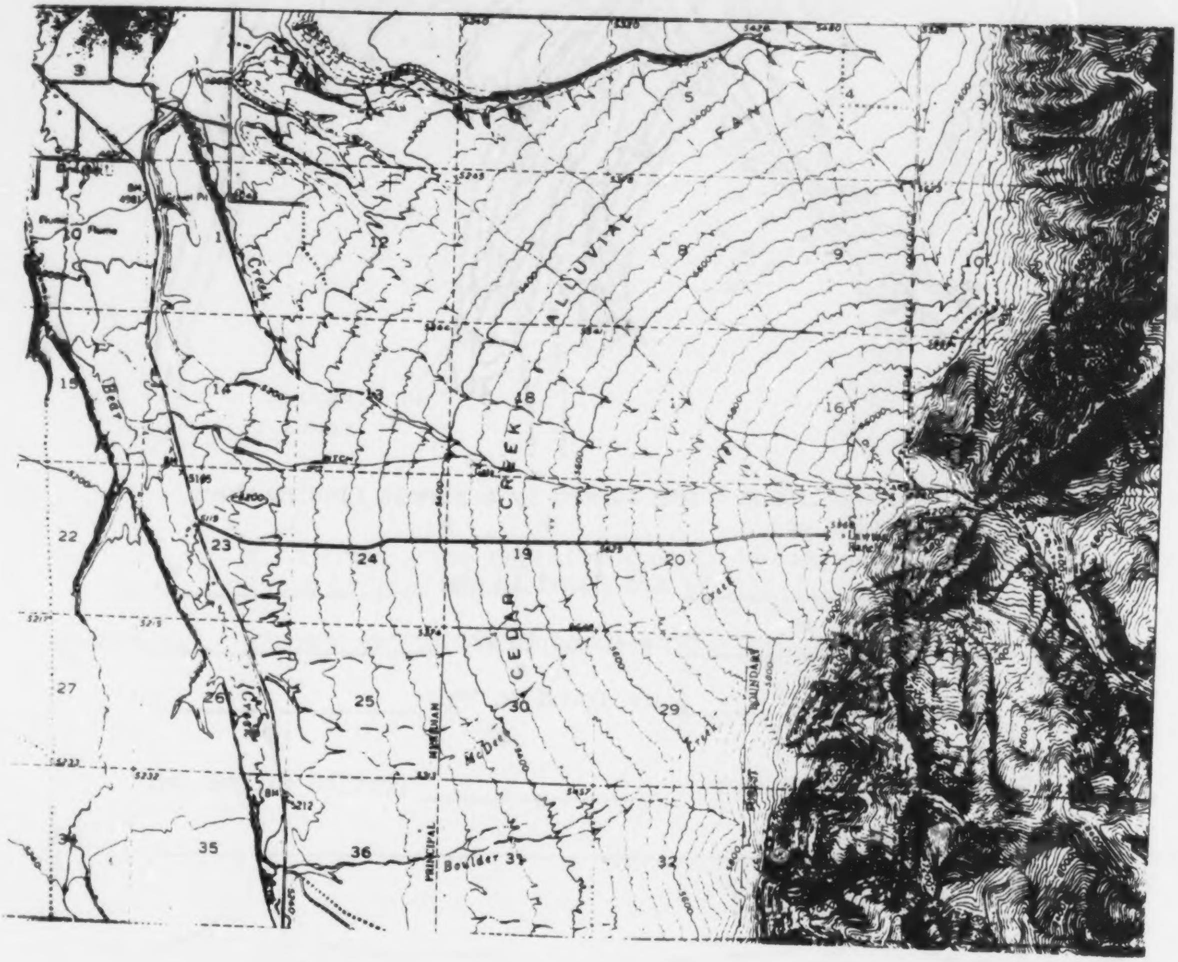
____________________ is found in the ____________________.

____________________ are found in the __________________.

Water is found in the _________________ portion of the fan

because ____.

FIGURE 1-7. ALLUVIAL FAN


CEDAR CREEK ALLUVIAL FAN
Bear
Creek
McDee
Boulder
PRINCIPAL MERIDIAN
BOUNDARY
3
4
5
9
10
12
15
16
18
19
20
22
23
24
25
26
27
29
30
32
35
36

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3. Glacial deposits - SEE FIGURE 1-8

a. Till - is anmixture of all sizes of material ranging from clay to boulders.

Till has _____ construction value!

b. Marginal moraines

(1) Terminal moraines - are ridges deposited at the glacier's furthest line of advance.
(2) Lateral moraines - are ridges deposited along the sides of a glacier.
(3) Medial moraines - are ridges deposited between the edges of two side-by-side glaciers.

________ is usually found in marginal moraines.

c. Ground moraines - are the deposits left by a glacier in the area between the marginal moraines.

These deposits are usually __________.

d. Drumlins - are tear-drop shaped hills.

Usually contain till deposits.

e. Eskers - are snake-like ridges left by rivers beneath a glacier.

_________ and ________ are usually found in eskers.

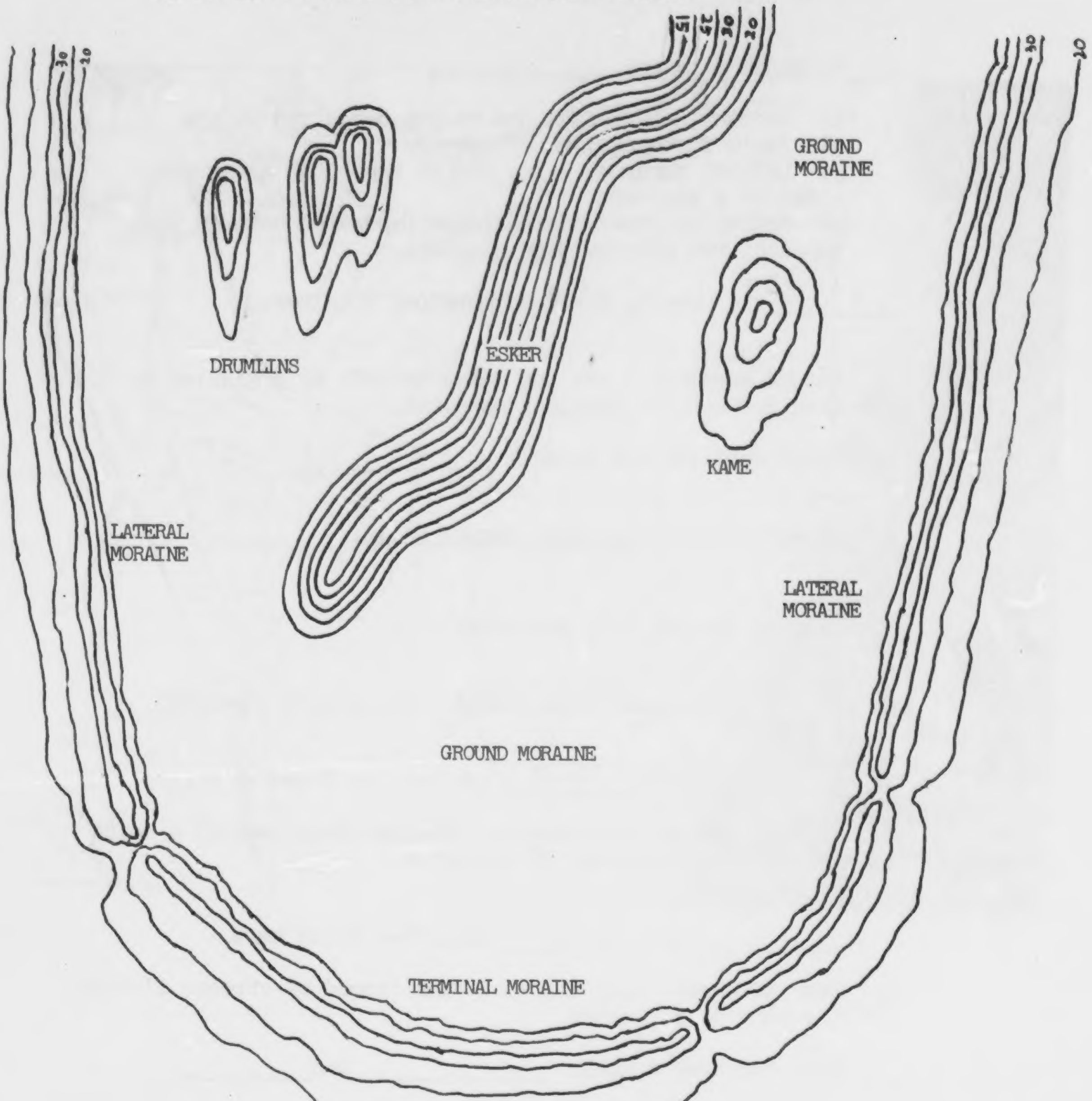
f. Kames - are hills formed by a stream which created a delta as it flowed off the edge of a glacier.

_________ and ________ are often found here.

g. Kame terraces - are alluvial fans formed by streams flowing off the edge of a glacier.

kame terraces contain ____________ and___________.

FIGURE 1-8.- GLACIAL AND FLUVIAL-GLACIAL FEATURES

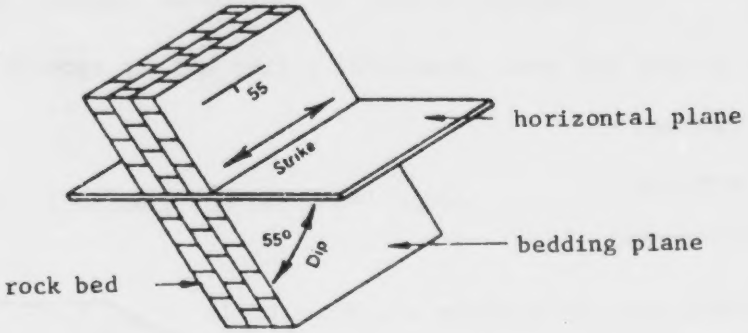

30
20
51
45
30
20
30
20
DRUMLINS
ESKER
GROUND MORAINE
KAME
LATERAL MORAINE
LATERAL MORAINE
GROUND MORAINE
TERMINAL MORAINE

4. Contour lines -

5. Quarries - are usually labeled on maps.

6. Strike and Dip - indicate positioning of rock layers. Can be determined by measurements taken from rock outcrops.

FIGURE 1-9. STRIKE AND DIP


55
Strike
horizontal plane
55°
Dip
bedding plane
rock bed

a. Strike and dip are important when:

   (1) drilling

   (2) blasting

   (3) quarrying

   (4) ripping

   (5) building roads into mountainsides

b. Measurement

(1) strike - is the azimuth of the line of intersection of the dipping layer and the horizontal plane.

FIGURE 1-10. STRIKE

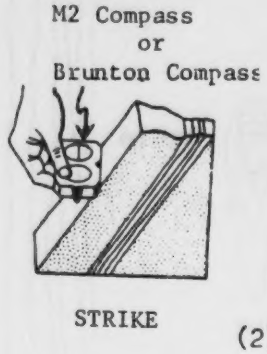
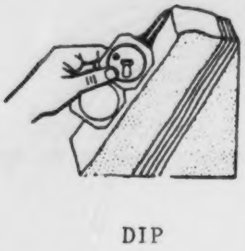


FIGURE 1-11. DIP



(2) dip - is perpendicular to the strike, and is the angle between the sloping layer and the horizontal plane.

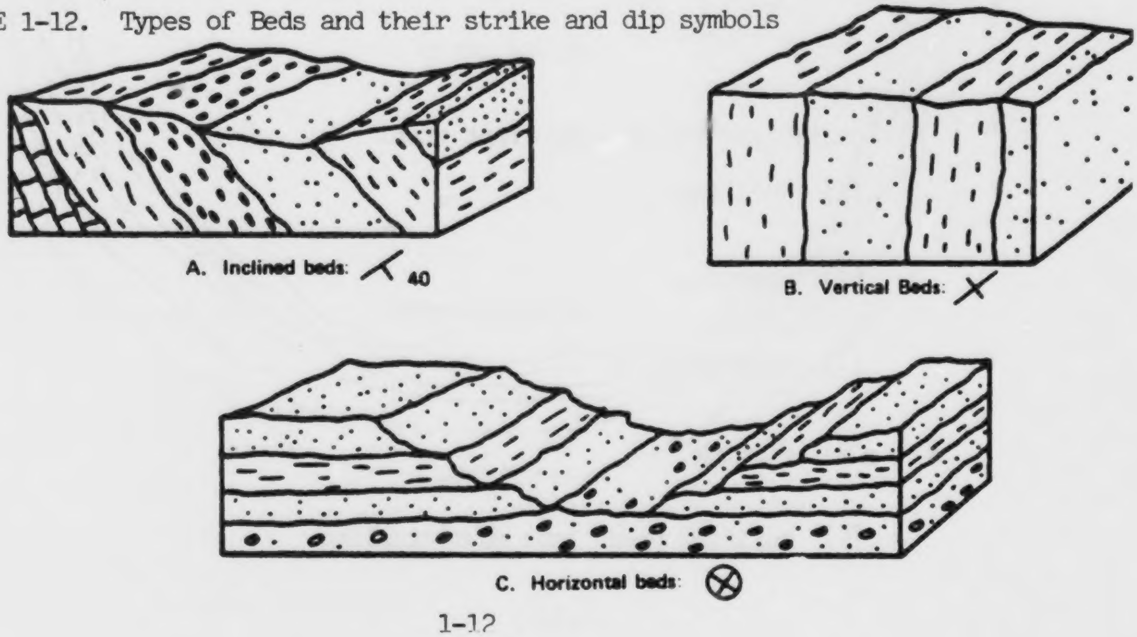
c. Types of beds and their associated strike and dip symbols.

(1) inclined -

(2) vertical -

(3) horizontal -

FIGURE 1-12. Types of Beds and their strike and dip symbols



A. Inclined beds: 40

B. Vertical Beds:

C. Horizontal beds: ⊗

C. Field Reconnaissance

1. Known Sites - are shown on maps. Examples include:

2. Probable Sites - are determined by preliminary map recon.

3. Possible Sites - are anywhere that just looks like it may be a useable source, but these are a LAST RESORT ONLY.

D. Record Data

1. Quality of Materials -

2. Quantity of Materials -

3. Location of each Site - 8-digit grid coordinate is required.

4. Factors affecting the ease of removal:

   (1)
   (2)
   (3)
   (4)

5. Equipment and personnel needed

PRACTICAL EXERCISES

Objective: To help the student understand how to:

(1) Locate and evaluate construction materials.
(2) Evaluate the military considerations of surface features.
(3) Evaluate the military significance of geologic structures.

Materials:

(1) Map packet, one per student or group of students.
(2) Pencil, paper, and straight-edge.

Instructions:

Students will solve each question presented in the problems. Students may be required to present their solutions to the class for critique.

EXERCISE #1. Map #1 - Aitkin, Minnesota.

Situation: Your combat engineer company has been given the mission to repair damaged railroad tracks running through the town of Aitkin. The company CP is located at Lone Pine School (R26W, T47N, Section 8). Your company commander has tasked you to locate a borrow pit site in order to obtain ballast for the railroad track.

Requirements:

(1) Identify three (3) likely sites to investigate for possible borrow pit operations.

1-
2-
3-

(2) Explain why you picked those sites.

EXERCISE #2. Map #2, Ennis Lake, Montana.

Situation: An aggressor army has invaded Canada, and after initial successes has halted near the U.S. - Canadian border. In anticipation of a counter-attack by allied forces, the 18th Engineer Brigade has tasked your unit to construct and/or improve airfield facilities in the Ennis Lake region. Initially your unit has been directed to improve the landing field in Section 10 on the southwest corner of Ennis lake to receive C-130 airplanes.

Requirements:

(1) Identify three (3) likely sites to recon for a source of good quality base course material for the runways.

1-
2-
3-

(2) Identify a likely site from which to obtain cohesive materials to use for a berm around the airfield fuel tanks.

(3) Justify your answers to (1) and (2) above.

EXERCISE #3. Map #3, Voltaire, North Dakota.

Situation: The allied counter-attack against the aggressor in Canada is continuing. Your company has been assigned the mission of upgrading the main supply route from the town of Voltaire north to Verendrye, a town on the next map.

Requirements:

(1) Identify two (2) likely sources of construction material south of the Souris River and two (2) other likely sources north of the river.

1-
2-
3-
4-

(2) How practical do you think cross-country movement is, in the center of this area? Why?

STUDENT NOTES

BASIC SOIL PROPERTIES

I. Soil - is a heterogeneous accumulation of uncemented or loosely cemented mineral grains inclosing voids of varying sizes. These voids may contain air, water, organic matter, or different combinations of these materials in varying amounts.

II. Reasons for studying soils.

A. Mobility/Countermobility

B. General engineering

III. Soil Formation

A. Parent materials

1. Sedimentary rocks

2. Igneous rocks

3. Metamorphic rocks

B. Weathering processes

1. Mechanical weathering - is where rocks are physically broken apart to form smaller pieces.

2. Chemical weathering - is where the materials making up the rocks changes its composition.

3. Result - Soil formation is a continuing process. The combination of different weathering processes, weathering rates, and parent materials has produced a large number of soils.

IV. Evaluation of Physical Characteristics

A. Particle Size

Soils are classified into four basic size groups under the Unified Soils Classification System (USCS).

1. Cobbles - are retained on a 3" sieve. The average diameter is between 3" and 10". Particles larger than 10" are not classified as soils but are boulders.

2. Gravels - will pass through a 3" sieve but are retained on a #4 sieve. The average diameter is between 5mm and 3".

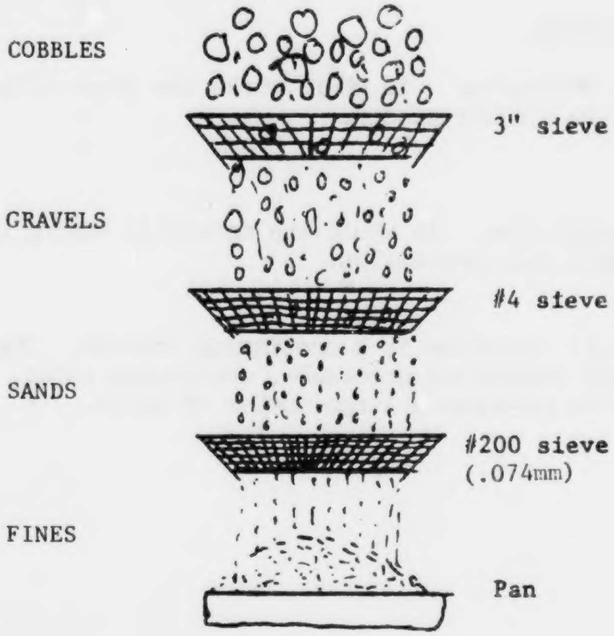
3. Sands - will pass through a #4 sieve but are retained on a #200 sieve. The average diameter is between .07mm and5mm.

4. Fines - will pass through the #200 sieve and are retained only on a solid pan. The average diameter is less than .07mm. Fines include both silt and clay.

   a. Silt - is bulky, non-plastic particles

   b. Clay - is particles that become plastic when mixed with water.

FIGURE 2-1. Sieves and Soil Sizes.



B. Particle Shape

1. Platy - where one dimension of a particle is considerably smaller than the other two dimensions.
   Suitability for construction - very poor

2. Bulky - where all three dimensions of the particle are relatively the same.

   Types:
   - a. rounded -

     poorly suited for construction

   - b. sub-rounded -

     fair for construction

   - c. sub-angular -

     good for construction

   - d. angular -

     excellent for construction

FIGURE 2-2. Platy shape

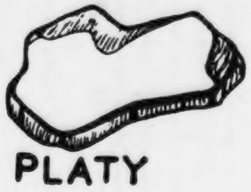
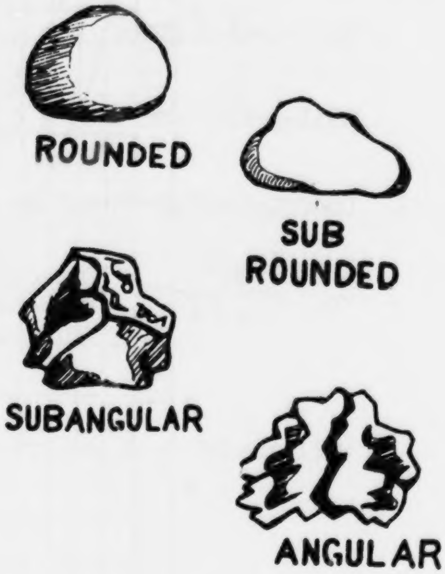


FIGURE 2-3. Bulky shapes



C. Gradation - is the relative amounts present of the different particle size groups within the soil.

1. Well graded -

2. Poorly graded -

    a. uniformly graded -

    b. gap graded -

D. Grain Size Distribution Chart (Figures 2-4 and 2-5)

1. Shape of the curve

2. Coefficient of uniformity (C_u)

$$C_u = \frac{D_{60}}{D_{10}} = \frac{\text{diameter of the 60\% passing size in millimeters}}{\text{diameter of the 10\% passing size in millimeters}}$$

Well-graded gravel has a C_u 4

Well-graded sand has a C_u 6

3. Coefficient of Curvature (C_c)

$$C_C = \frac{(D_{30})^2}{D_{60} \times D_{10}} = \frac{(\text{Diameter of the 30\% passing size in millimeters})^2}{(\text{Diameter of the 60\% passing size in millimeters}) \times (\text{Diameter of the 10\% passing size in millimeters})}$$

Well-graded sand and gravel have C_c = 1 to 3 (inclusive)

FIGURE 2-4. Grain size distribution chart

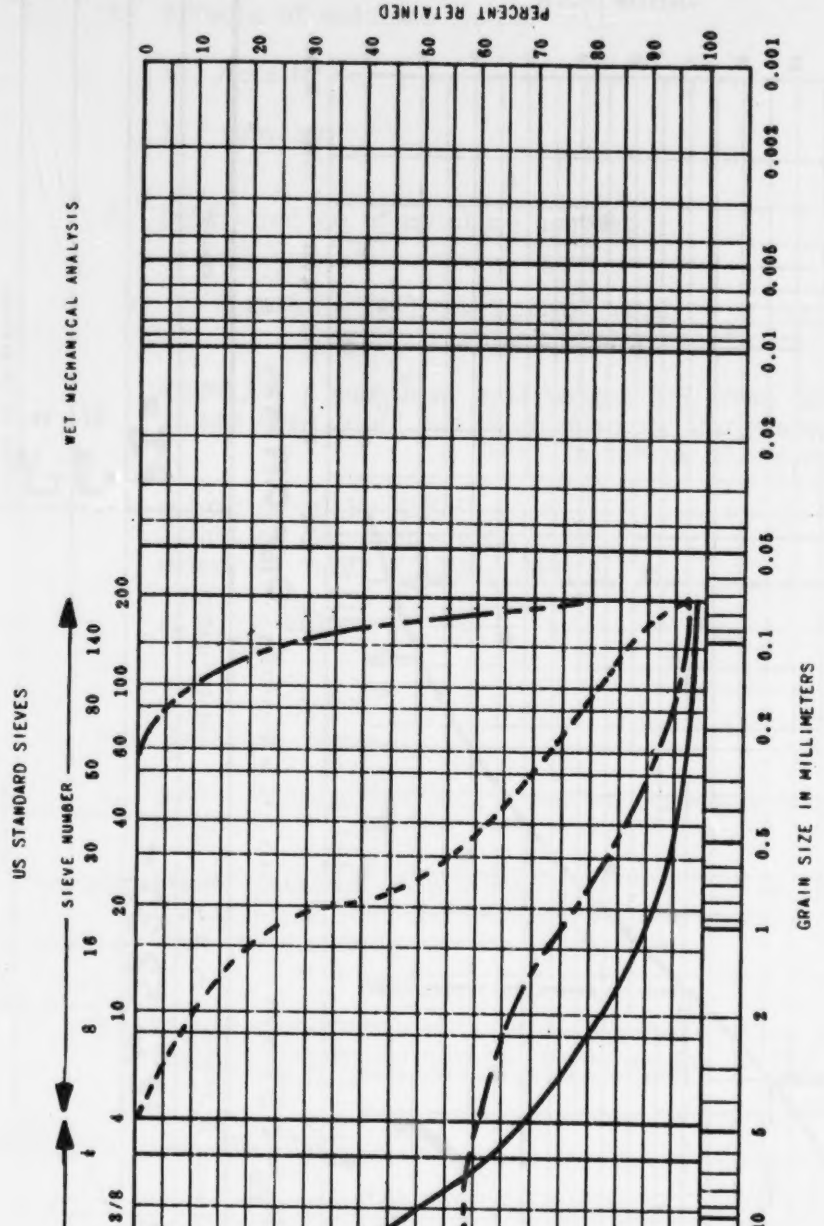
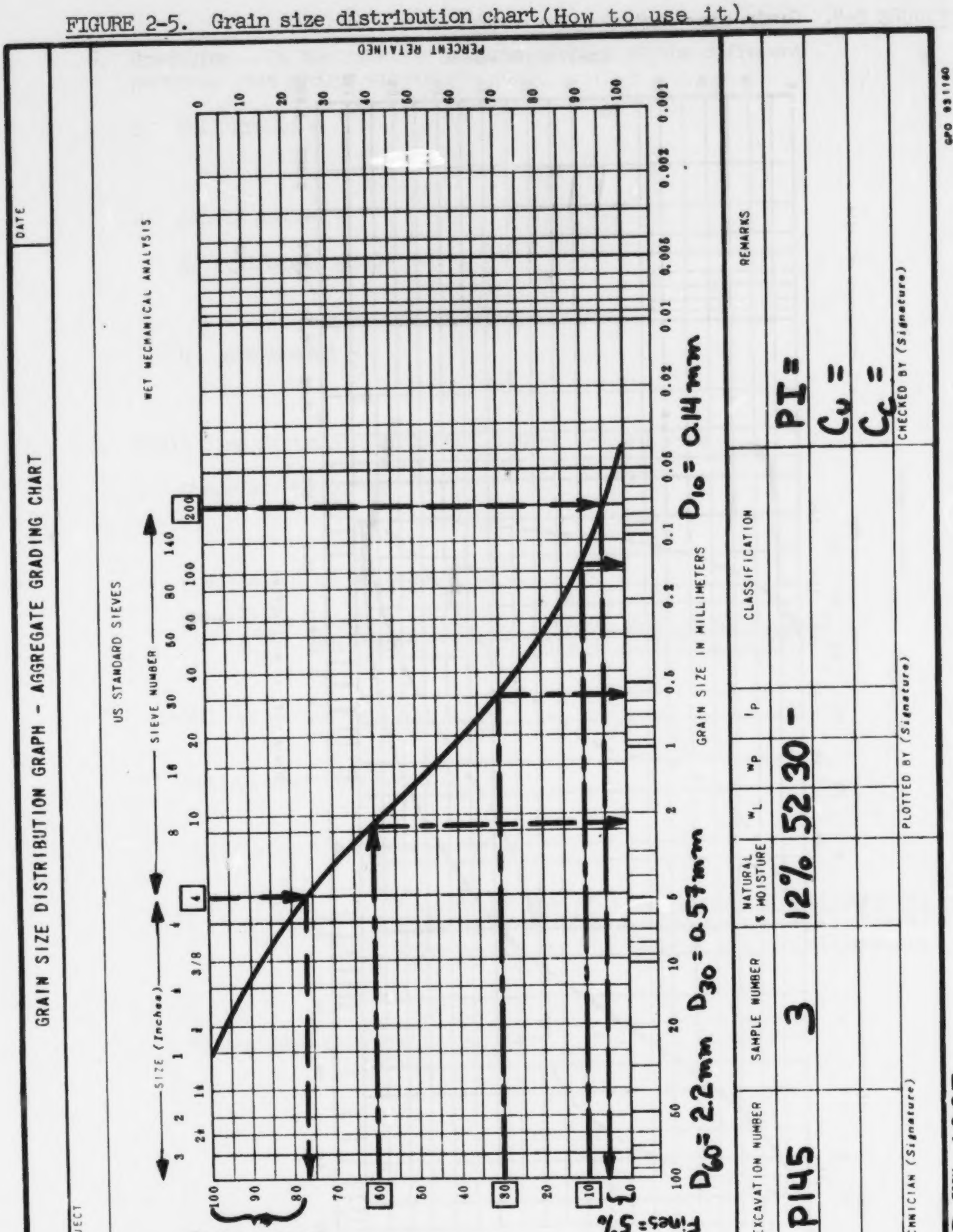


FIGURE 2-5. Grain size distribution chart(How to use it).



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS
P145	3	12%	52	30	-		PI =
							C_u =
							C_c =

E. Soil Moisture

1. Effects of moisture in the soil.

   a. density

   b. cohesion

2. Determination of moisture content

$$\% \text{ water} = (w\%) = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

EXAMPLE: A sample of soil weighs 225 grams (g), and after it has been dried out it weighs only 200 grams. What is the w% of the soil sample?

weight of soil = 225g
weight of dry soil = 200g
weight of water = 225g - 200g = 25g

$$w\% = \frac{25g}{200g} \times 100 = 12.5\%$$

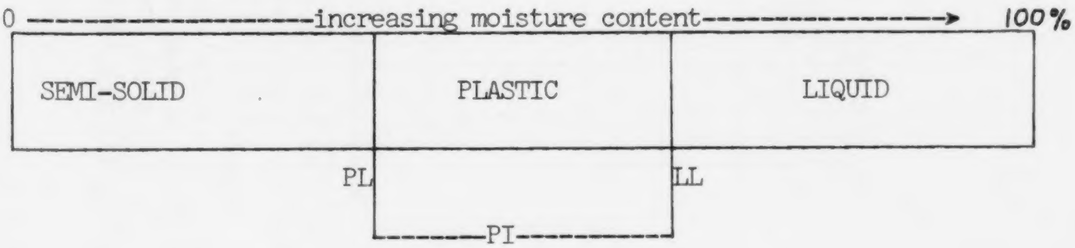
F. Soil Moisture States

1. Semi-solid state - soil crumbles under a deforming pressure or load.

2. Plastic state - When a deforming pressure or load is applied, the material will deform and remain in that deformed shape when the applied pressure is released.

3. Liquid state - the soil is wet enough to flow under its own weight without any outside force acting on it.

FIGURE 2-6. SOIL MOISTURE STATES



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G. Plasticity Chart (SEE Figure 2-7)

1. Liquid Limit =LL - is the moisture content at which a soil begins to have liquid properties.

2. Plastic Limit = PL - is the moisture content at which a soil begins to have plastic properties.

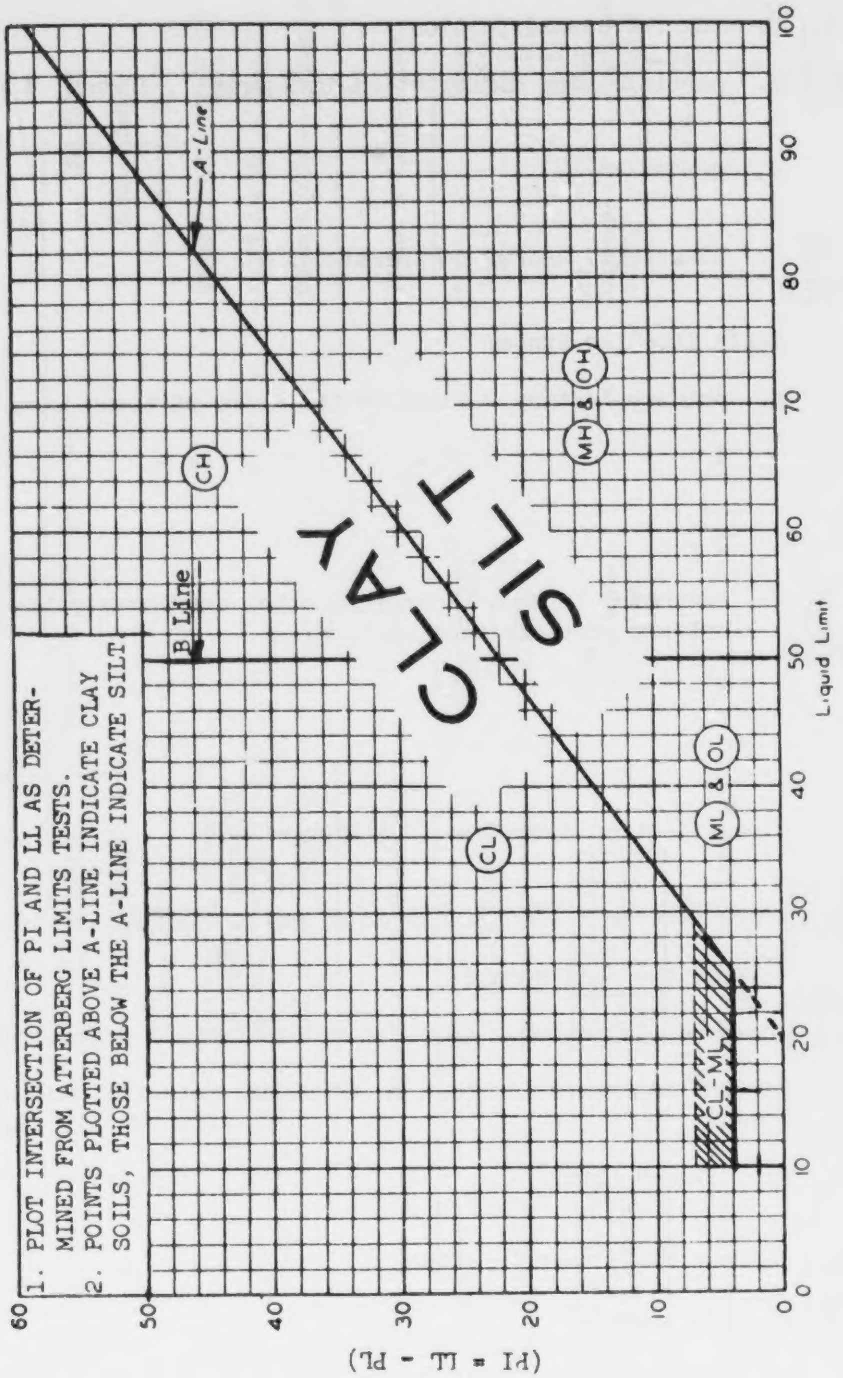
3. Plasticity Index = PI - is the measure of the range of the plastic state.

   PI = LL - PL

4. The "A" Line
   a. Silts -
   b. Clays -

5. The "B" Line
   a. High compressibility -
   b. Low compressibility -

FIGURE 2-7. PLASTICITY CHART



1. PLOT INTERSECTION OF PI AND LL AS DETERMINED FROM ATTERBERG LIMITS TESTS.
2. POINTS PLOTTED ABOVE A-LINE INDICATE CLAY SOILS, THOSE BELOW THE A-LINE INDICATE SILT

V. Soil Classification

A. Unified Soils Classification System (USCS)

1. Grounds for classification

    a. particle size distribution in % gravel, % sand, % fines.

    b. gradation

    c. plasticity and/or compressibility

2. Soils labeling system

    a. primary letters - identify the major particle size portion.

    G -

    S -

    M -

    C -

    O -

    b. secondary letters - describe other characteristics of the primary particle size portion.

    M -

    L -

    W -

    C -

    H -

    P -

3. Classification procedure (SEE Figure 2-8)

B. Procedure for field identification of soils (SEE Soils Field ID Car

1. Collect the soil sample

2. Separate soil using a #4 sieve and estimate % gravel retained.

3. Perform sedimentation test on #4(minus) material and estimate % sand.

4. Calculate % fines

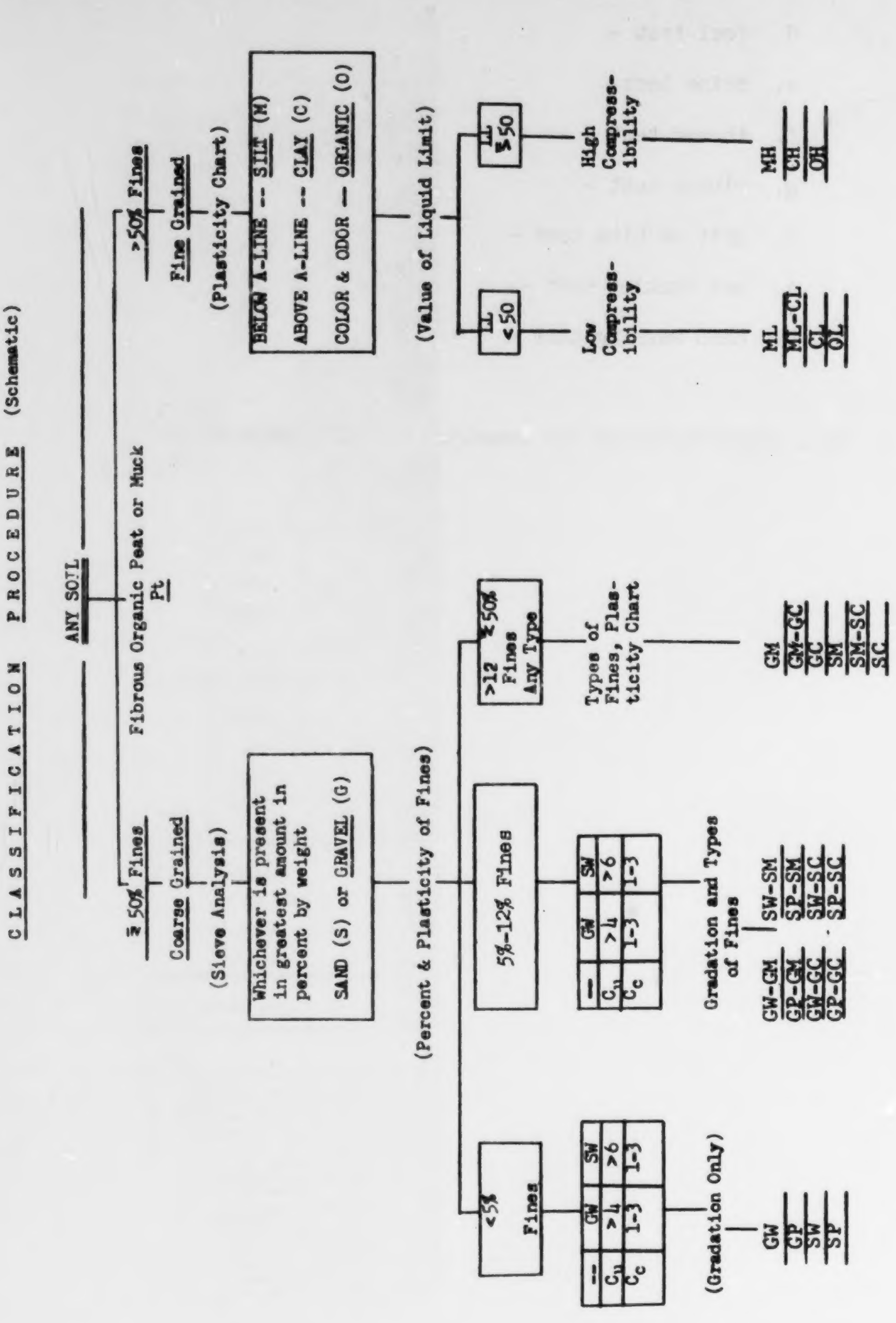
5. Perform field tests on fines

    a. odor test -

    b. breaking(dry strength) test -

    c. powder test -

FIGURE 2-8. Schematic for Soils Classification Procedure


CLASSIFICATION PROCEDURE (Schematic)
ANY SOIL
≤ 50% Fines
Coarse Grained
(Sieve Analysis)
Whichever is present in greatest amount in percent by weight
SAND (S) or GRAVEL (G)
(Percent & Plasticity of Fines)
<5% Fines
— GW SW
Cu >4 >6
Cc 1-3 1-3
(Gradation Only)
GW
GP
SW
SP
5%-12% Fines
— GW SW
Cu >4 >6
Cc 1-3 1-3
Gradation and Types of Fines
GW-GM
GP-GM
GW-GC
GP-GC
SW-SM
SP-SM
SW-SC
SP-SC
>12 ≤50% Fines Any Type
Types of Fines, Plasticity Chart
GM
GM-GC
GC
SM
SM-SC
SC
Fibrous Organic Peat or Muck
Pt
>50% Fines
Fine Grained
(Plasticity Chart)
BELOW A-LINE -- SILT (M)
ABOVE A-LINE -- CLAY (C)
COLOR & ODOR — ORGANIC (O)
(Value of Liquid Limit)
LL <50
LL ≥50
Low Compressibility
High Compressibility
ML
ML-CL
CL
OL
MH
CH
OH

d. feel test -

e. shine test -

f. thread test -

g. ribbon test -

h. grit or bite test -

i. wet shaking test -

j. hand washing test -

C. Soil characteristics for construction (SEE Table-1)

TABLE 1 Soil Characteristics for Construction

Major Divisions		Letter (3)	Symbol		Name (6)	Value as Foundation When Not Subject to Frost Action (7)	Value as Base Directly under Bituminous Pavement (8)	Potential Frost Action (9)	Compressibility and Expansion (10)	Drainage Characteristics (11)	Compaction Equipment (12)	Unit Dry Weight Lb Per Cu Ft (13)	Field CBR (14)	Subgrade Modulus k_g Lb Per Cu In. (15)
(1)	(2)		Hatching (4)	Color (5)										
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	○	Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125-140	60-80	300 or more
		GP	●		Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	110-130	25-60	300 or more
		GM d	▥	Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	130-145	40-80	300 or more
		u				Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40	200 to 300
		GC	▨		Clayey gravels, gravel-sand-clay mixtures	Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40	200-to 300
	SAND AND SANDY SOILS	SW	⁘	Red	Well-graded sands or gravelly sands, little or no fines	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110-130	20-40	200 to 300
		SP	⁘		Poorly graded sands or gravelly sands, little or no fines	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100-120	10-25	200 to 300
		SM d	▥	Yellow	Silty sands, sand-silt mixtures	Good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired equipment sheepsfoot roller; close control of moisture	120-135	20-40	200 to 300
		u				Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20	200 to 300
		SC	▨		Clayey sands, sand-clay mixtures	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20	200 to 300
FINE GRAINED SOILS	SILTS AND CLAYS LL < 50	ML	▥	Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	100-125	5-15	100 to 200
		CL	▨		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Fair to poor	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100-125	5-15	100 to 200
		OL	▦		Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90-105	4-8	100 to 200
	SILTS AND CLAYS LL > 50	MH	▥	Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller	80-100	4-8	100 to 200
		CH	▨		Inorganic clays of high plasticity, fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller	90-110	3-5	50 to 100
		OH	▨		Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller	80-105	3-5	50 to 100
HIGHLY ORGANIC SOILS		Pt	☰	Orange	Peat and other highly organic soils	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical	-	-	-

Notes:

1. Column 3, Division of GM, and SM groups into subdivisions of d and u are for roads and airfields only; subdivision is on basis of Atterberg limits; suffix d (e. g., GMd) will be used when the liquid limit is 28 or less and the plasticity index is 6 or less; the suffix u will be used when the liquid limit is greater than 28.
2. Column 7, values are for subgrades and base courses except for base course directly under bituminous pavement.
3. In column 8, the term "excellent" has been reserved for base materials consisting of high quality processed crushed stone.
4. In column 9, these soils are susceptible to frost as indicated under conditions favorable to frost action described in the text.
5. In column 12, the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled. In some instances, several types of equipment are listed, because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.
   a. Processed base materials and other angular materials. Steel-wheeled rollers are recommended for hard angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.
   b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. Equipment size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:
      Crawler-type tractor -- total weight in excess of 30,000 lb.
      Rubber-tired equipment -- wheel load in excess of 15,000 lb, wheel loads as high as 40,000 lb may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi).
      Sheepsfoot roller -- unit pressure (on 6- to 12-sq-in. foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be necessary to obtain the required densities for some materials. The area of the feet should be at least 5 per cent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
6. Column 13, unit dry weights are for compacted soil at optimum moisture content for modified AASHO compactive effort.

D. PRACTICAL EXERCISES FOR FIELD IDENTIFICATION

1. This soil is fine grained and non-organic. It has a positive reaction to the wet shaking test and cannot be rolled into a 1/8" diameter thread at any moisture content, but a 2" ribbon can be formed. All of this soil passes the #40 sieve.

What is the symbol for this soil?______

2. Approximately 40% of this soil does not pass a #4 sieve. The material that passed the #4 sieve is given the sedimentation test and 60% of it settles out in 30 seconds. When you test the material that passed the #40 sieve you find that it shines and tastes smooth but also reacts quickly to the wet shaking test.

What is the symbol for this soil?______

3. When sieved, approximately 22% of this soil fails to pass the #4 sieve. The material that passed the #4 sieve is given the sedimentation test and approximately 70% of it settles out in 30 seconds. The material that passes the #40 sieve does not shine when shine tested. This material will only form a 2" ribbon and reacts very positively to the wet shaking test.

What is the symbol for this soil?_______

4. Sixty (60) % of this soil passes the #4 sieve. In a sedimentation test, 80% of this sub-#4 material settles out. The material that passes the #40 sieve feels harsh and irritating but will roll into a 1/8" thread. As you clean up, the material is very easy to wash off of your hands.

What is the symbol for this soil?______

5. Forty-five (45)% of this soil is gravel and 60% of the remaining material settles out in a sedimentation test. A dry pat of this soil breaks easily and feels harsh and irritating. The sub-#40 portion of this soil will not form a thread.

What is the symbol for this soil?______

6. This soil is 10% gravel, 15% sand, and 75% fines. It will form a ribbon over 10" long, and a dry pat of it can not be broken.

What is the symbol for this soil?______

7. Five (5)% of this soil will not pass tht #4 sieve. A sedimentation test shows that 99% of the sub-#4 material settles out in 30 seconds. The sub-#40 material (fines) will not form a thread or a ribbon. Your 51G tells you that the soil has a C_u of 10 and a C_c of 1.8.

What is the field symbol for this soil?____

What is the USCS symbol for this soil?_____

8. Which of the seven soils you identified above would be suitable for use in the following situations?

a. Which of the soils would be best for constructing an impermeable berm around fuel storage points?

____

b. Which soils would you use to help insulate a pavement from the effects of freezing and thawing?

____

c. Which soils would you use to fill sandbags?

____

d. Which soils would you use as a filter to strain small organic particles from waste water as you return it to the water table?

____

STUDENT NOTES

SOIL COMPACTION

I. Compaction - is the process of increasing the density of a soil by squeezing air out of voids, while not significantly changing the moisture content of the soil.

II. Reasons for compaction

A.

B.

C.

D.

III. Laboratory test procedures

A. California Bearing Ratio (CBR) test -(SEE Chapter 4 of this book)

B. Proctor Mold Tests

IV. Compaction Curves

A. Curve development

1. Wet soil density = $(\gamma_w) = \frac{\text{wet weight of sample}}{\text{volume of sample}}$ = density in $\frac{\text{lbs}}{\text{ft}^3}$

2. Moisture content $(w) = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100\%$

weight of water = wet sample weight - dry sample weight

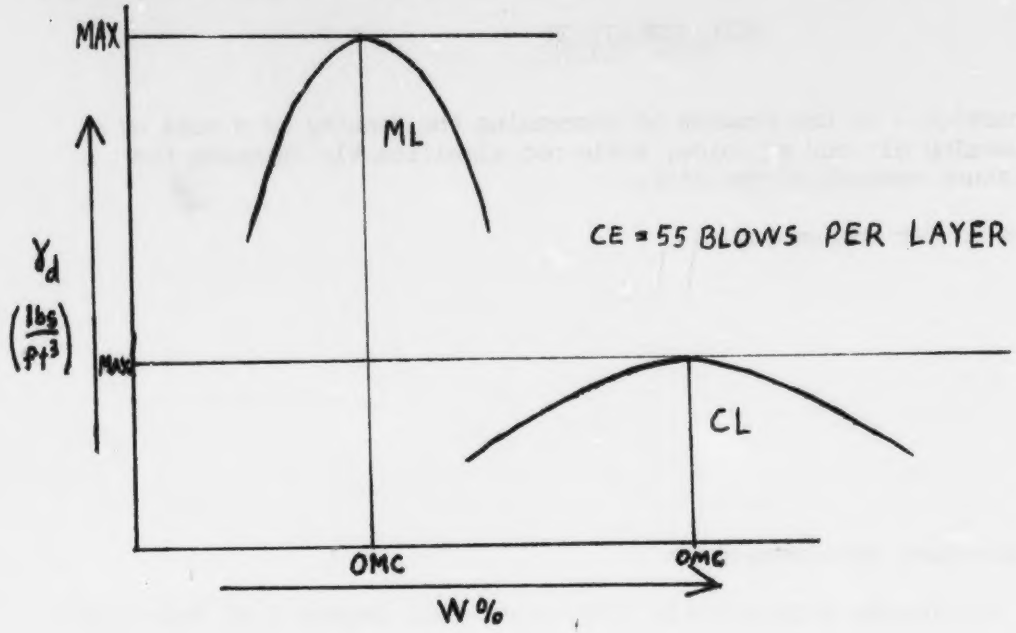
3. Dry soil density = $(\gamma_d) = \frac{\text{dry weight of sample}}{\text{volume of sample}}$ = density in $\frac{\text{lbs}}{\text{ft}^3}$

B. Characteristic curve shapes for CE 12, 26, 55, 72.

1. Coarse-grained soils -

2. Fine-grained soils -

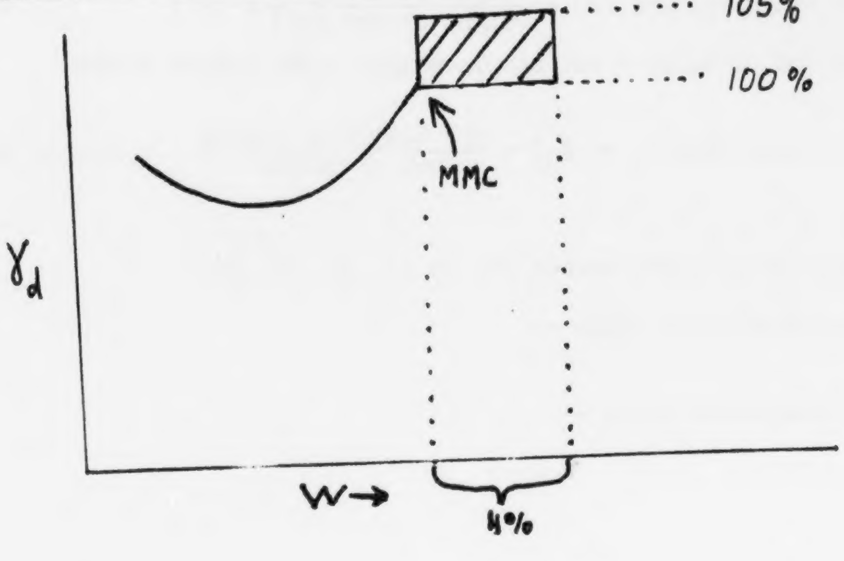
FIGURE 3-1. Sample Compaction Curve For A Fine-Grained Soil



C. Optimum Moisture Content (OMC) -

D. Maximum Dry Density for a given Compactive Effort-

FIGURE 3-1.5. Sample Compaction Curve for a Free-Draining Soi

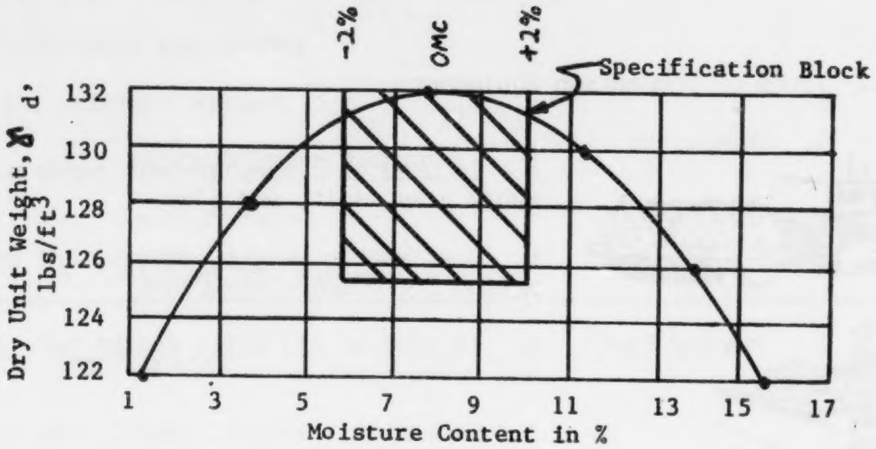


E. Specification Block for Non-cohesive Soils

1. Dry Density (γ_d) Limits

2. Moisture Content (w%) Limits

FIGURE 3-2. Spec Block for Non-cohesive Soils

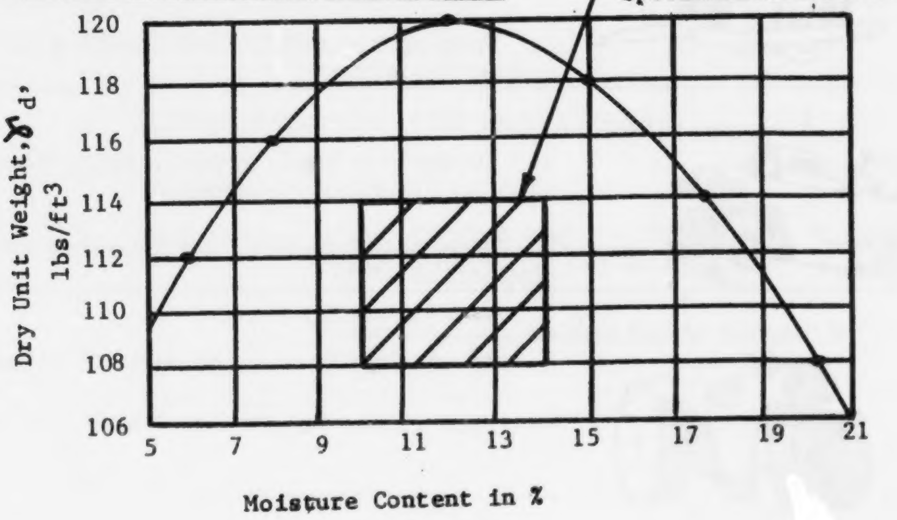


F. Specification Block for Cohesive Soils

1. Dry Density (γ_d) Limits

2. Moisture Content (w%) Limits

FIGURE 3-3. Spec Block for Cohesive Soils



V. Compaction Equipment

A. Types of Compactive Effort

1. Static weight -

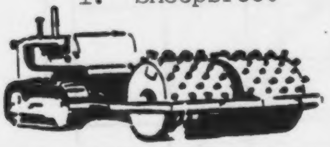
2. Impact -

3. Vibration -

4. Kneading action -

B. Types of Compaction Equipment

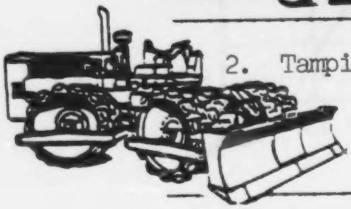
1. Sheepsfoot



a. Is used on clayey gravel, sandy silt, sandy clay, silt, and clay.

b. Uses static weight, impact, and kneading action for compactive-effort.

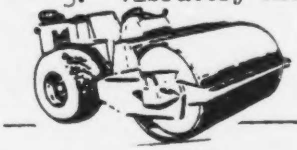
2. Tamping Foot



a. Is used on all soils except "pure sand" and "pure clay".

b. Uses static weight, impact, and kneading action for compactive effort.

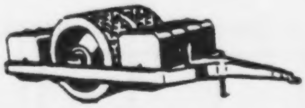
3. Vibratory Roller



a. Is used on sand, gravel, and gravel mixtures.

b. Uses vibration and static weight for compactive effort.

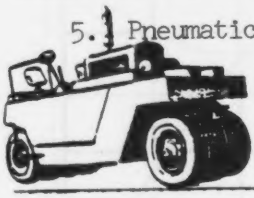
4. Grid Roller



a. Is used on rock, large granular fill, and glacial till.

b. Uses static weight and kneading action for compactive effort.

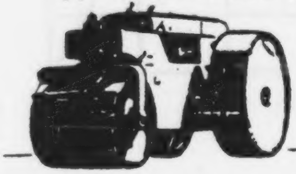
5. Pneumatic Roller



a. Is used on sand, gravel, silt, clay, and also on bituminous pavements.

b. Uses static weight and kneading action for compactive effort.

6. Steel Wheel Roller



a. Is used on gravel and also on bituminous

b. Uses static weight for compactive effort

VI. Field Density Testing

A. Expedient procedures

1. Find moisture content using wet and dry densities.
    a. SEE IV.A on page 3-1 of this workbook.
    b. Dry soil by heat, alcohol burn-off, or air-drying.

2. Estimate density by observing groundswell around tires of heavy vehicles and the amount of tire penetration.

B. Deliberate procedures

1. Moisture content

    a. Speedy moisture tester -

    b. Oven-drying -

    c. Nuclear densometer -

2. Soil density

    a.

    b.

    c.

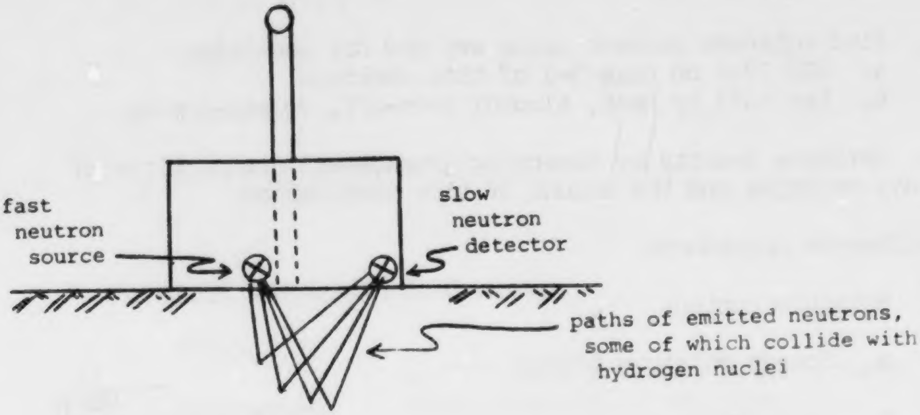
    d.

    e. Nuclear densometer (SEE Figure 3-4)

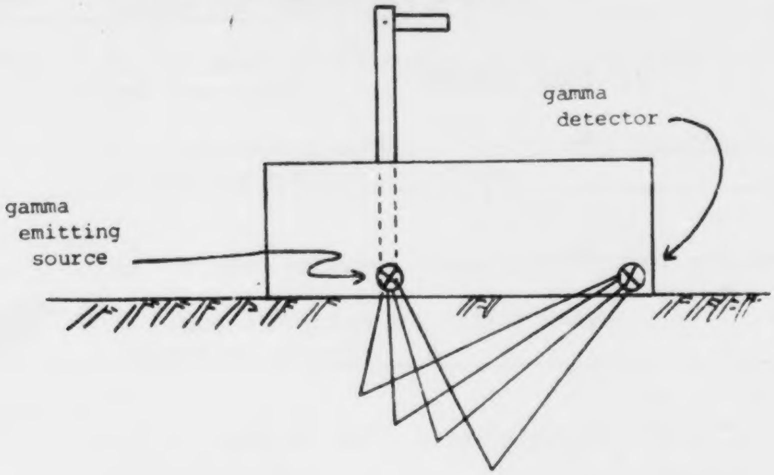
    f. Sand cone: $\frac{\text{weight of sand used to fill hole}}{\text{known density of the sand}} = \text{volume of soil}$

$$\frac{\text{weight of soil}}{\text{volume of soil}} = \text{density of the soil}$$

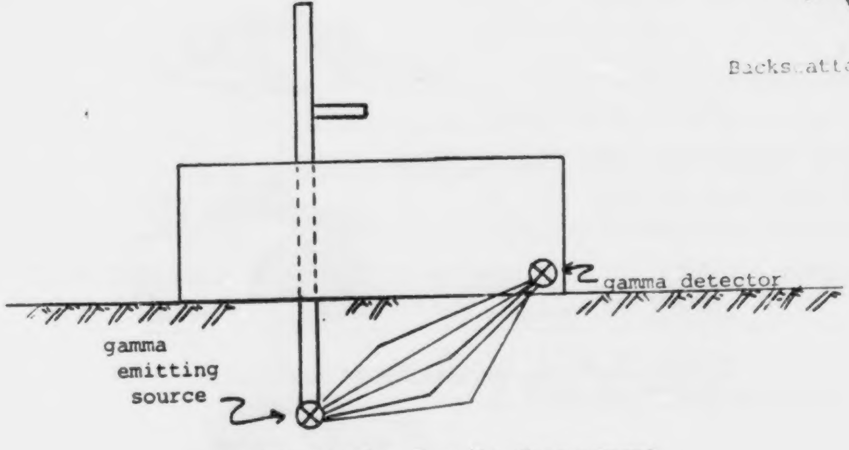
FIGURE 3-4. CAMPBELL PACIFIC NUCLEAR CPN MC-1


fast
neutron
source
slow
neutron
detector
paths of emitted neutrons,
some of which collide with
hydrogen nuclei

Backscatter Moisture Measurement


gamma
detector
gamma
emitting
source

Backscatter Density Measurement


gamma detector
gamma
emitting
source

Transmission Density Measurement

Practical Exercise for Sand Cone Method of field density determination

1. Situation: You are in the theater of operations at an airfield which is under construction. You check the density of the compacted base course using the sand cone method and obtain the following data.

2. You have the following facts from which you must calibrate the sand that you will use in the sand cone test.

   a. volume of the mold = 1/30 cubic feet (cf)
   b. weight of the mold = 9.5 lbs.
   c. weight of sand plus mold = 12.63 lbs
   d. density = $\frac{\text{weight}}{\text{volume}} = \frac{\text{lbs}}{\text{cf}}$ = pcf

   What is the density of the sand?___________ pcf

3. You take a soil sample from the compacted sub-base at the airfield. The wet soil weighs 7.62 lbs. Using the alcohol burn-off method you determine that the moisture content is 13.6%. It takes 4.83 lbs of the calibrated sand to fill the hole from which you took your soil sample.

   a. What is the volume of your soil sample?_____________

      note: volume = $\frac{\text{weight of sand}}{\text{density of sand}}$

   b. What is the wet density of your sample?_____________

      Note: wet density = $\frac{\text{weight of wet sample}}{\text{volume}}$

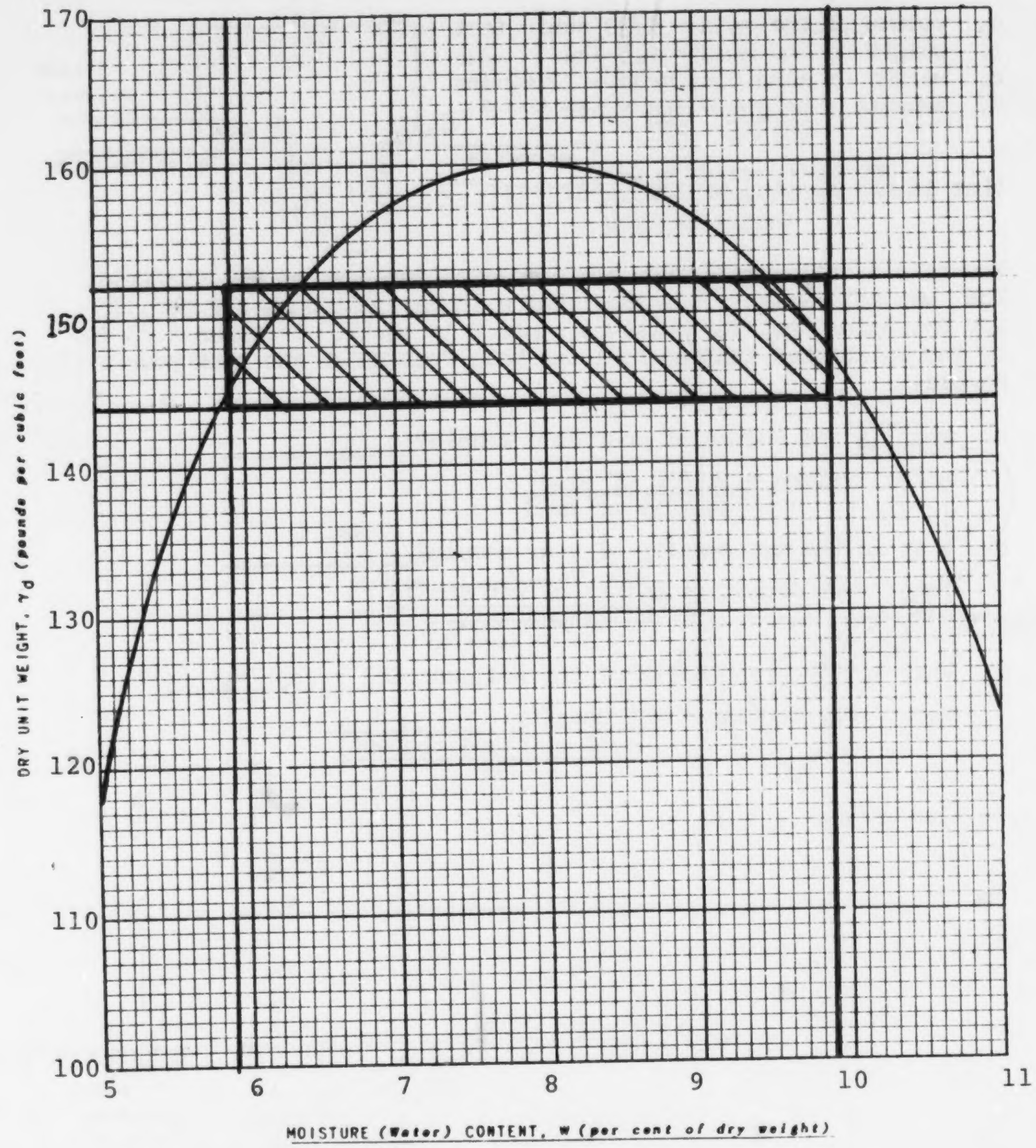
   c. What is the dry density of the soil sample?_____________

      Note: dry density = $\frac{\text{wet density}}{1 + \left(\frac{\text{\% moisture content}}{100}\right)}$

VII. Corrective Actions

If the moisture content and density of a soil do not fall within the spec block, corrective measures must be taken to ensure that they are within the correct range.

FIGURE 3-7. Corrective actions for incorrect moisture content or density.



Practical exercise for Figure 3-8.

List the corrective actions to be taken when your soil has the following soil moisture contents and densities.

1. $w = 8.6\%$ ACTIONS:
   $\gamma_d = 100$pcf

2. $w = 4.2\%$ ACTIONS:
   $\gamma_d = 106$pcf

3. $w = 11.2\%$ ACTIONS:
   $\gamma_d = 107$ pcf

4. $w = 6.2\%$ ACTIONS:
   $\gamma_d = 109$pcf

5. $w = 6.2\%$ ACTIONS:
   $\gamma_d = 117$pcf

6. $w = 8.4\%$ ACTIONS:
   $\gamma_d = 111$pcf

7. $w = 11.8\%$ ACTIONS:
   $\gamma_d = 101.5$ pcf

8. $w = 4.5\%$ ACTIONS:
   $\gamma_d = 103$ pcf

FIGURE 3-8.

SOIL COMPACTION TEST GRAPH

DATE

PROJECT

RESULTS

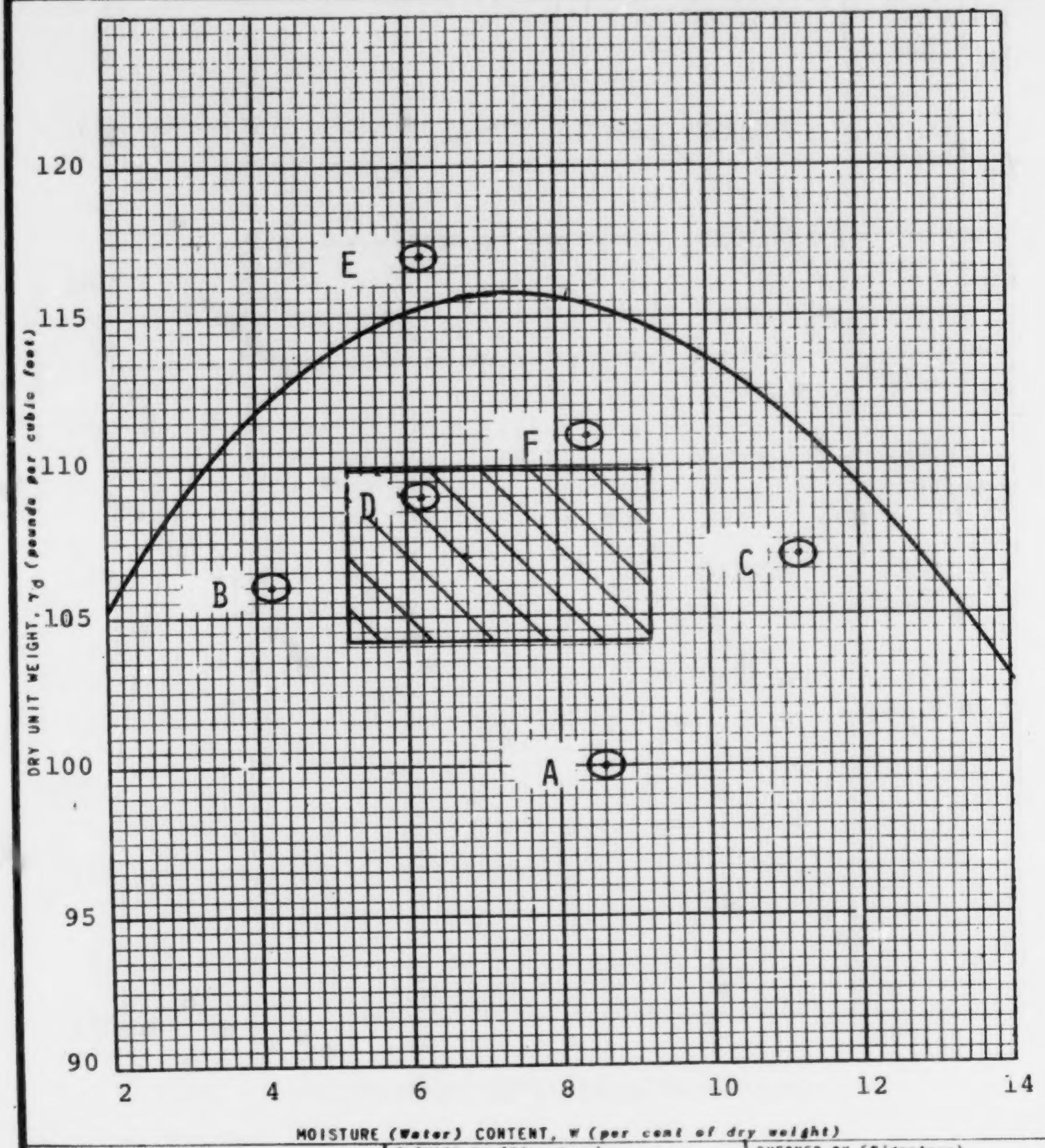
MAXIMUM DRY UNIT WEIGHT

EXCAVATION NUMBER

SAMPLE NUMBER

REMARKS *(Use reverse side, if more space is needed)*

OPTIMUM MOISTURE *(Water)* CONTENT



TECHNICIAN *(Signature)*

PLOTTED BY *(Signature)*

CHECKED BY *(Signature)*

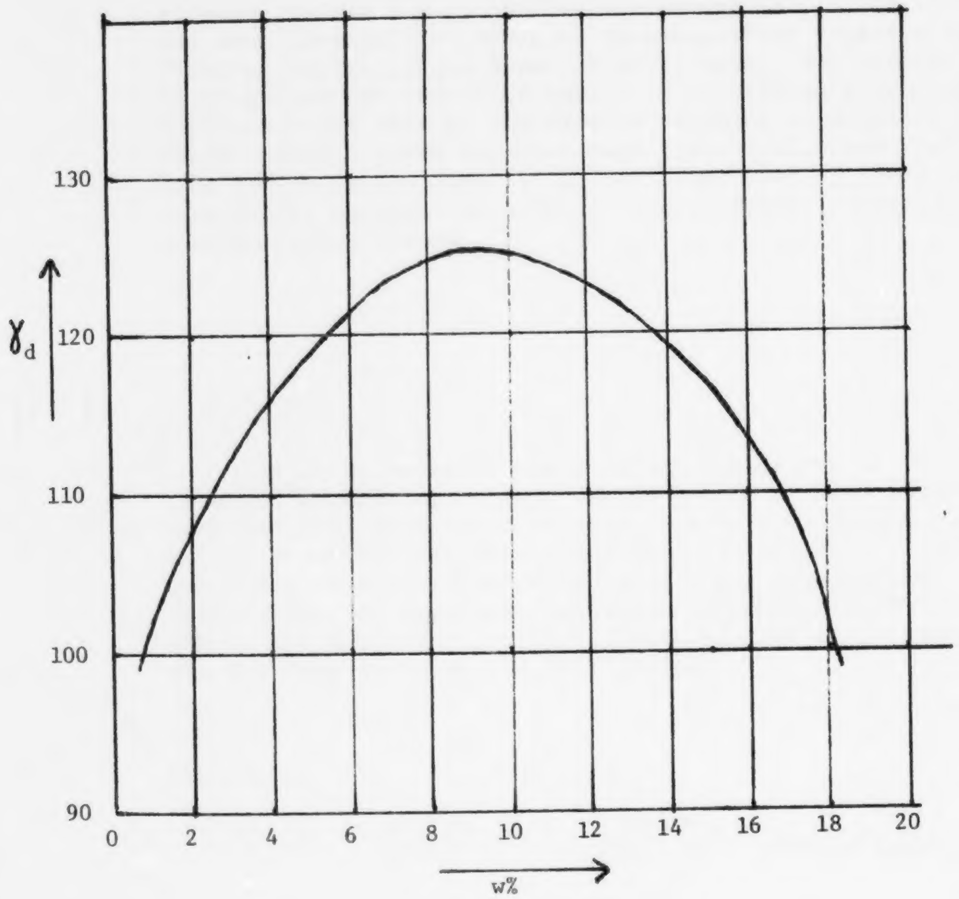
DD FORM 1 AUG 57 **1211**

Practical exercise for Figure 3-9.

Answer the following questions about this soil.

1. What is the Optimum Moisture Content? ______________
2. What is the CE55 maximum dry density? ______________
3. What moisture range should be used in the field? ___________% to ________%
4. What are the acceptable limits of density after compaction? _____ to _____pcf
5. Draw the spec block on Figure 3-9.

FIGURE 3-9.



STUDENT NOTES

CALIFORNIA BEARING RATIO

INTRODUCTION

A. History:

During the early 1940's, the responsibility for the design and construction of military roads and airfields was assigned to the Corps of Engineers. Later, the Corps was delegated the same function pertaining to civil airfields and roads required for military use. After deliberation, the principles then used by the California Highway Department, known as California Bearing Ratio Method (CBR) were adopted for the design of flexible pavements. Since 1940, the Corps has been involved in continued investigations relative to Flexible Pavement Design in the areas of development, validation, and revision. Although the CBR method of design is considered a semi-empirical approach where each of the factors relating to a soil's ability to resist traffic loads have not been individually analyzed, its worth as a design method lies in the fact that over 35 years of experience with actual pavement structures have provided the basis for our present design curves.

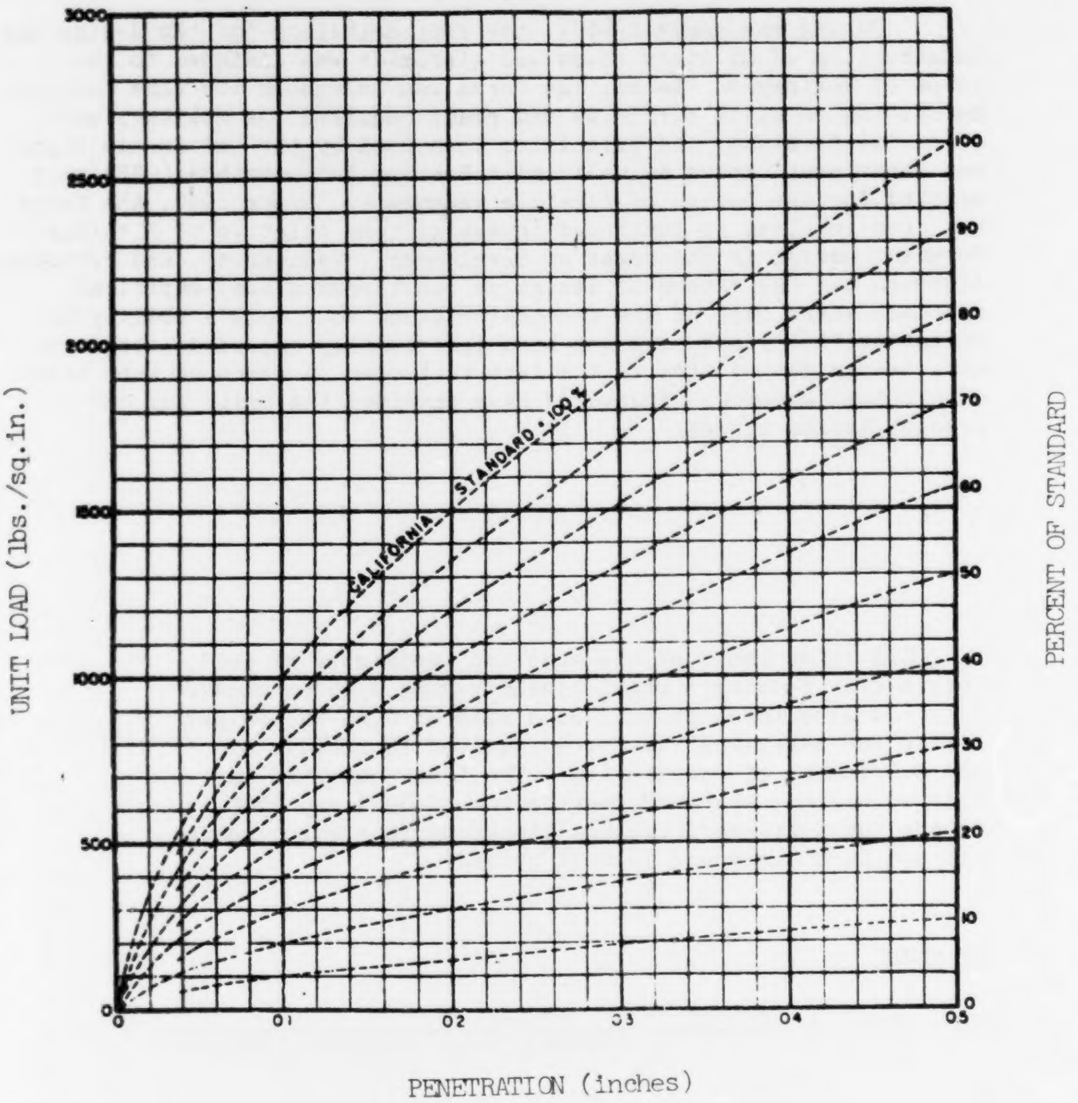
CBR is an index of the shearing resistance of soil, obtained by forcing a steel piston having a three square inch end area into the soil at a rate of 0.05 inches per minute and measuring the force required to achieve 0.1 and 0.2 inches of penetration. The force obtained is ratioed against standard test values of 3000 and 4500 pounds for a standard crushed limestone (one which passes the 3/4 inch sieve and is well graded).

EQUATION 1

$$\text{CBR RATIO} = \frac{\text{ACTUAL SOIL STRESS}}{\text{STRESS OF STANDARD (-3/4" crushed limestone)}} \times 100$$

FIGURE 4-1.

CALIFORNIA BEARING RATIO TEST GRAPH
(Plot test curve below to obtain corrected unit load)



To illustrate the computations required to obtain the CBR value for a soil, the results of two typical penetrations tests shown on Figure 2 will be used. Curve "A" is representative of results from a test in which all went well. Although the stress vs. penetration plot is not linear, it does approvimate this condition at penetration depths less than 0.30 inches. At larger pentration depths values, the curve has a decreasing slope indicating plastic failure of the soil. The CBR value is computed by reading the stress values at 0.1 and 0.2 inches penetration and use of equation 1 as follows:

$$CBR(0.1) = (200 / 1000) \times 100 = 20\%$$

$$CBR(0.2) = (345 / 1500) \times 100 = 23\%$$

The next question that arises is which value should be used References state : "the larger of the two CBR values are used and should CBR(0.2) be largest it must be verided by testing another sample."

This selection criteria implies that:

(1) Strengths used to design a pavement will be measured at no more than 0.2 inch of soil deformation.

(2) If CBR (0.1) is greater than CBR (0.2) the soil has begun plastic failure between these limits of deformation and soil strengths obtained at plastic failure will not be used.

(3) If CBR (0.2) is greater then CBR(0.1), the soil is still undergoing elastic deformation up to 0.2 inches of penetration and that this elastic strength can be used of verified by second test.

In the preceding example, the appropriate CBR would be 23% if verified by a second test.

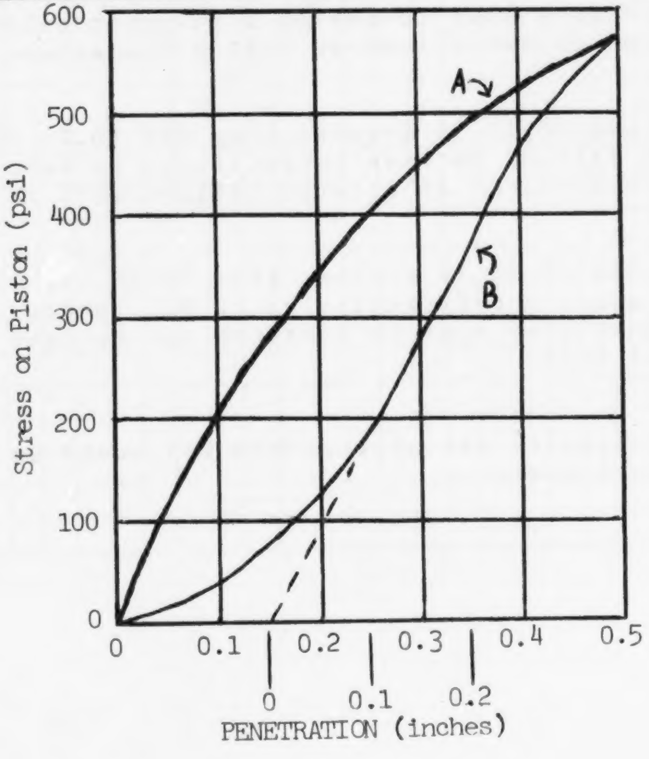
Curve "B" in Figure 2 which has a concaved upward shape between 0 and 0.23 inches penetration is a manifestation of surface irregularities resulting from the lack of complete initial contact between the piston and the soil sample surface. Compensation for this fault can be made by extending the straight line portion of the stress/penetration curve until it intersects the abscissa and taking that intersection as the new origin. The stresses at the new 0.1 and 0.2 in. penetrations are shown in Figure 3. The CBR values are computed by epuations 3 and 4. Selection of the specific CBR (i.e. CBR(.1) of CBR(.2) are made using the same guides as for curve "A". In the example:

3) CBR(.1) = (190 /1000) x 100 = 19%

4) CBR(.2) = (375 /1500) x 100 = 25%

The appropriate value would be 25% if the test were verified. After a CBR value has been determined from the penetration data, that value by convention is referred to as a corrected CBR whether of not adjustment to the origin has been made.

FIGURE 4-2. TYPICAL CBR PENETRATION TEST RESULTS



FLEXIBLE PAVEMENT CONSIDERATIONS

Subgrade - The in-place natural soil which supports the pavement structure. It may have a CBR value ranging from almost 0 to 100% and specifications may require that portions of it must be compacted.

Select Material - A naturally occurring imported soil having CBR values less than 20% but greater than the compacted subgrade. It is used to carry stress levels intermediate to those supported by the subgrade and subbase material.

Subbase Material - Imported cohesionless material having CBR values ranging from 20% to 50%. It is normally obtained from natural gravel deposits and serves to carry intermediate stresses in the pavement.

Base Course - Highest quality structural material having strengths near that of the CBR standard material. For pavement design, CBR values are normally assigned based on gradation requirements rather than CBR tests. Base courses are always cohesionless and are usually processed to obtain the desired gradation.

Asphalt Concrete - Material that serves primarily as a wearing course and moisture barrier.

FIGURE 4-3

3 in. Asphalt Concrete	
6 in. Base Course	CBR = 100 @ 100 to 105% CE55 w 5%
10 in. Subbase	CBR = 40 @ 100 to 105% CE55 w = 8 to 12%
6 in. Select	CBR = 18 @ 95 to 100% CE55 w = 15 to 19%
Compacted Subgrade	CBR = 10 @ 95 to 100% CE55 w = 15 to 19%
Uncompacted Subgrade	CBR = 5 @ 80% CE55 w = 21%

SELECTION OF A TEST PROGRAM

The type of test program and the analysis of the resulting data depends to a considerable degree on where the soil is going to be used in the pavement, whether or not the soil will be disturbed during the construction process, the level of compactive effort required to preclude settlement, and the physical nature of the soil. From Figure it can be seen that the materials above the uncompacted subgrade will undergo considerable disturbance and the laboratory tests used to define strength must reflect this fact. Likewise, a test method that gives strengths for undisturbed samples should be used for the uncompacted subgrade. The effect of density must be included in the evaluation of compactive effort corresponding to those required by compaction specifications. Lastly, the nature of the soil will determine the amount and type of laboratory work required. For example, coarse-grained, free-draining soils or those soils having few fines may display little strength variation with changes in moisture content while some fine-grained soils show objectionable volumetric changes (swelling) as the molding moisture content varies. Table 2, page , is a summary of the test methods which should serve as a guide for the following discussion. However, the basic principle underlying this categorization is simulation of the conditions to which a soil is subjected during both construction and the life of the structure.

The following facts should be kept in mind when developing a test program:

1. An increase in compactive effort produces an increase in density and a decrease in OMC (where OMC is the optimum moisture content at a specified compaction).

2. The relationship of strength, water content, and density is complex but follows a distinct pattern. Knowledge of this pattern is essential to intelligent design.

3. Variations of blows in laboratory tests and variations of passes, foot size, tire pressure, lift thickness, and gross load produce variations in compactive effort. Proper application of 1. and 2. above can predict the effect of the variations.

The engineer may modify the programs described in this text to fit the special needs of a particular project.

IN-PLACE TESTS

A. FIELD CBR TESTING

As a part of flexible pavement design, it will be necessary to determine the strength of the subgrade soil at its natural density. To accomplish this, the CBR penetration apparatus can be mounted on a vehicle from which the penetration test can be performed in a manner similar to the laboratory method (see Figure 3). In-place tests are used for design under any one of the following conditions:

(1) The in-place density and moisture content are such that the degree of saturation is 80%.

(2) The soil is coarse grained and cohesionless so that it is not effected by changes in water content (free-draining soils).

(3) The soil has been in place for several years so that fluctuation in moisture content are minimal (a minimum of three years in which the soil is undisturbed is required). This applies to road construction over an old fill.

B. UNDISTURBED SAMPLE* TESTING

A shortcoming of the field CBR test is that the swell of a soil cannot be monitored. Additionally, the loss of strength on saturation cannot be determined. If the in-place strength of a soil thought to be expansive is required, or if moisture changes will effect the soil, an undisturbed sample can be taken by attaching a sampling collar to a CBR mold and forcing it into the soil. This test should only be used with cohesive soils.

*Driving the CBR mold into the soil will actually cause some disturbance. Care must be taken to insure that this disturbance is held to a minimum. This is a misnomer - it is really nonremolding testing.

FIGURE 4-4.

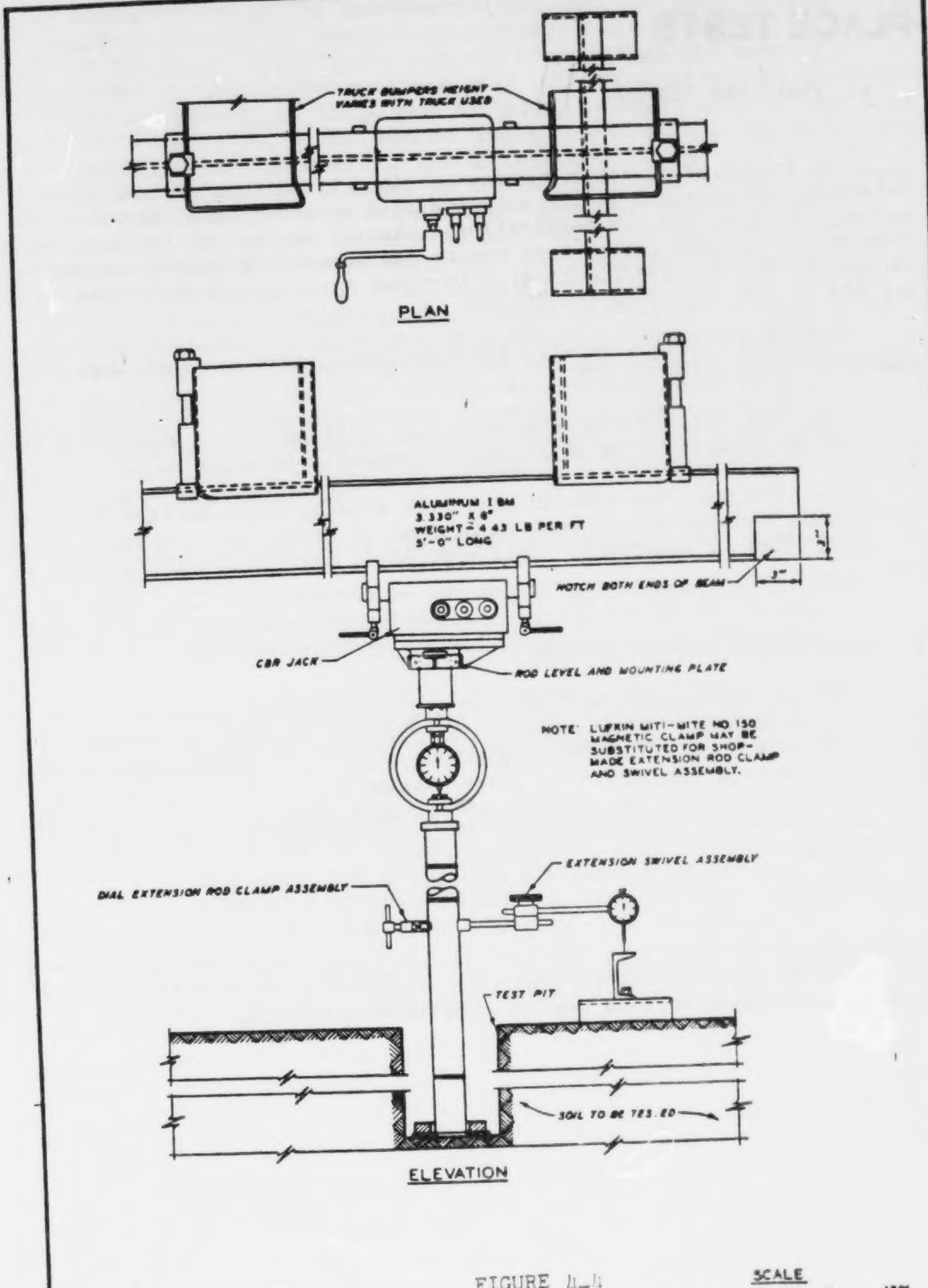

TRUCK BUMPERS HEIGHT VARIES WITH TRUCK USED
PLAN
ALUMINUM I BM
3.330" X 6"
WEIGHT = 4.43 LB PER FT
5'-0" LONG
3"
3"
NOTCH BOTH ENDS OF BEAM
CBR JACK
ROD LEVEL AND MOUNTING PLATE
NOTE: LUFKIN MITI-MITE NO 150 MAGNETIC CLAMP MAY BE SUBSTITUTED FOR SHOP-MADE EXTENSION ROD CLAMP AND SWIVEL ASSEMBLY.
EXTENSION SWIVEL ASSEMBLY
DIAL EXTENSION ROD CLAMP ASSEMBLY
TEST PIT
SOIL TO BE TESTED
ELEVATION
SCALE
4
0
4
8
12 IN.
03:780-A

FIGURE 4-4

EXPEDIENT TESTS

The tactical situation in the Theather of Operations may preclude a full range of CBR tests as described above. The engineer may reduce the amount of testing or alter some of the test procedures as long as he understands the effect of changes on the test results and the limitations of using results from tests which are not performed in the standard manner. The test methods described below present some methods of determining CBR values. These procedures are listed in order from least accurate to most accurate. The engineer should choose the most accurate method available based on time and equipment constraints. The success of the mission is dependent upon how the CBR measurements depict the actual strength of the soil.

(1) Correlation to USGS soil classification. Table 2-4 TM 5-330 (Table 3 in this text) gives ranges of CBR values which correlate to each soil type (USGS classification). When no testing method is available the lower value in this range may be taken as a very approximate CBR value. However, no design of permanent or semi-permanent pavement should be done without testing. These values might serve as a guide for preliminary planning.

(2) Airfield Index. TM 5-330 details the use of the cone penetrometer of determine an airfield index. Generally the process involves pushing a penetrometer (contained in the soils trafficability test set) into the ground and reading the resistance of the soil to penetration at various depths. Readings should be taken from the penetrometer gage at even depths (i.e. 2,4,6,etc) down to 24 inches. The dial reading is called the cone index and is converted to an airfield index by dividing it by the cone end area (in hundreds of an inch). For example:

airfield index = cone index / 50 for the 0.5 in cone

or airfield index = cone index / 20 for the 0.2 in cone

The airfield indexes are averaged to give a single value at one location. This average airfield index can be converted to CBR for design purposes through the use of Figure 2-17 TM 5-330 shown below. These values generally are conservative. Penetrometer tests should be made randomly along the length of a proposed runway. The lowest CBR value computed would then be used for design.

FIGURE 4-5.

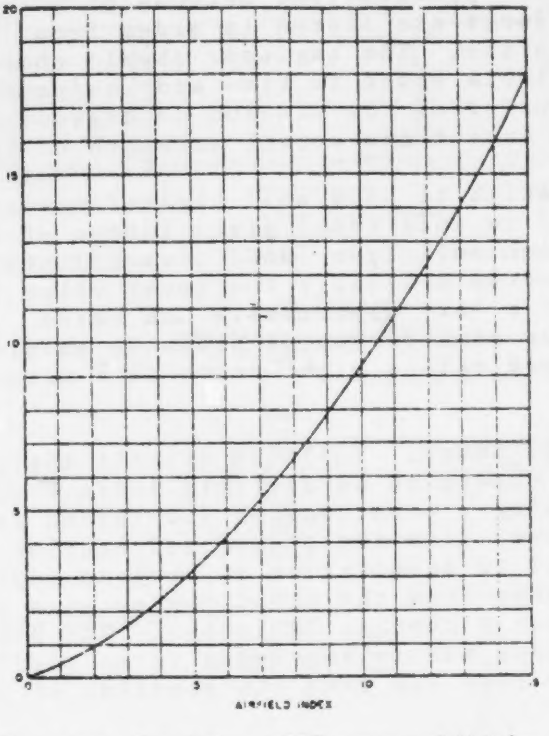


Figure 2-17. Correlation of CBR versus airfield index.

(3) Simulated In-Place CBR

The first step in simulated in-place CBR is to determine the most likely conditions to be expected. This can range from soaked to arid conditions, and is determined by an analysis of samples taken from the site, and climate of the area. The best likely projection must be made. Both moisture content and dry density determinators are important, and are taken from an undisturbed sample, or the largest chunk available. Volumes may have to be estimated when necessary.

Once the in-place conditions have been determined, the analyst must attempt to duplicate them in the laboratory with the remainder of the sample. Moisture content can be controlled through accurate addition of water, while estimates must be made of density. Two samples may be necessary for density duplication, trying one compactive effort, and adjusting based upon what density is calculated.

CBR testing should be conducted on the duplicated sample, using normal penetration procedures. The analyst must ensure that appropriate boxes on the worksheet (DD From 1212) be marked for soaked or unsoaked conditions. The CBR value determined will approximate the strength of the soil when in the same state as that of the sample.

Further testing may be conducted on the tested sample to approximate anticipated conditions. This predicted condition may be a field observation or long range forcast.

(4) One Point Test

This test follows the same procedures as described previously, except that only one mold is compacted. This is done in the following manner:

(a) Determine the optimun moisure content and CE55 maximum density.
(b) Compact one mold at this density and OMC.
(c) Soak and penetrate the mold in the standard way.
(d) Record the CBR value.

It should be noted that this method does not take into account variations in field compaction since the soil will never be compacted exactly at OMC and max dry density on the job site. If saturation is not anticipated, the sample can be penetrated dry, then soaked and penetrated on the opposite end under soaked conditions. This will provide a comparison of saturated vs. moist strengths of the soil.

(5) Three Point CBR

This test follows the same procedures used in the Corps of Engineers method except that only three molds are compacted and penetrated. Generally the test consists of the following variations on the Corps of Engineers method:

(a) Determine the optimum moisture content for the soil and CE 55 mixmum dry density.

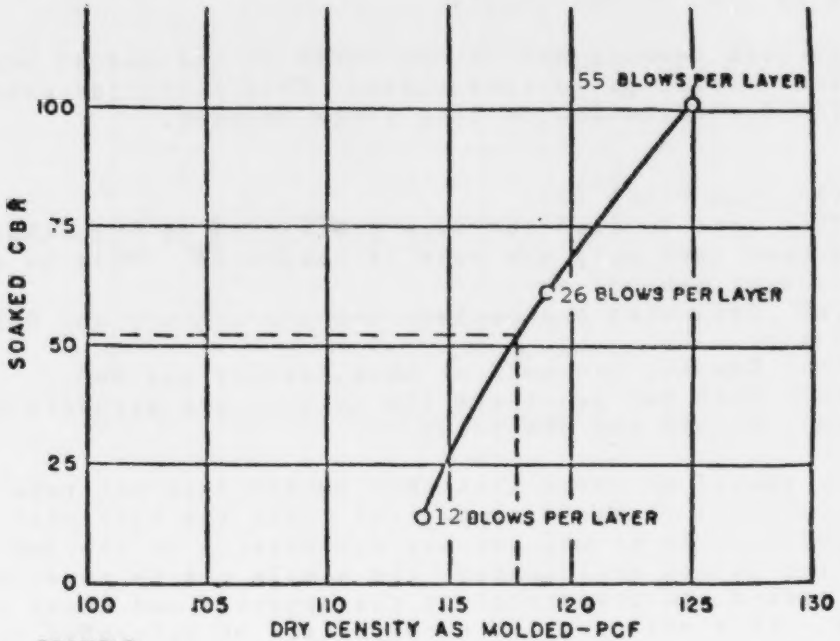
(b) Compact three CBR molds at optimum moisture content varying the compactive effort for each (i.e. 12, 26, 55 blows/layer).

(c) Soak the samples and penetrate the molds in the standard way.

(d) Plot the soaked CBR values vs. dry density.

(e) Find the CBR which corresponds to the minimum acceptable dry density for the project (i.e. 96% CE55 maximum dry density) as shown below.

FIGURE 4-6.



EXAMPLE

GIVEN: MAXIMUM DRY DENSITY BY CE55 METHOD = 124 P.C.F.
FIND: THE CBR AT 95 PERCENT OF THE ABOVE MAXIMUM DRY DENSITY.
SOLUTION: 95 PERCENT OF 124 P.C.F. = 117.8 P.C.F.
AT 117.8 P.C.F., THE CBR IS 52

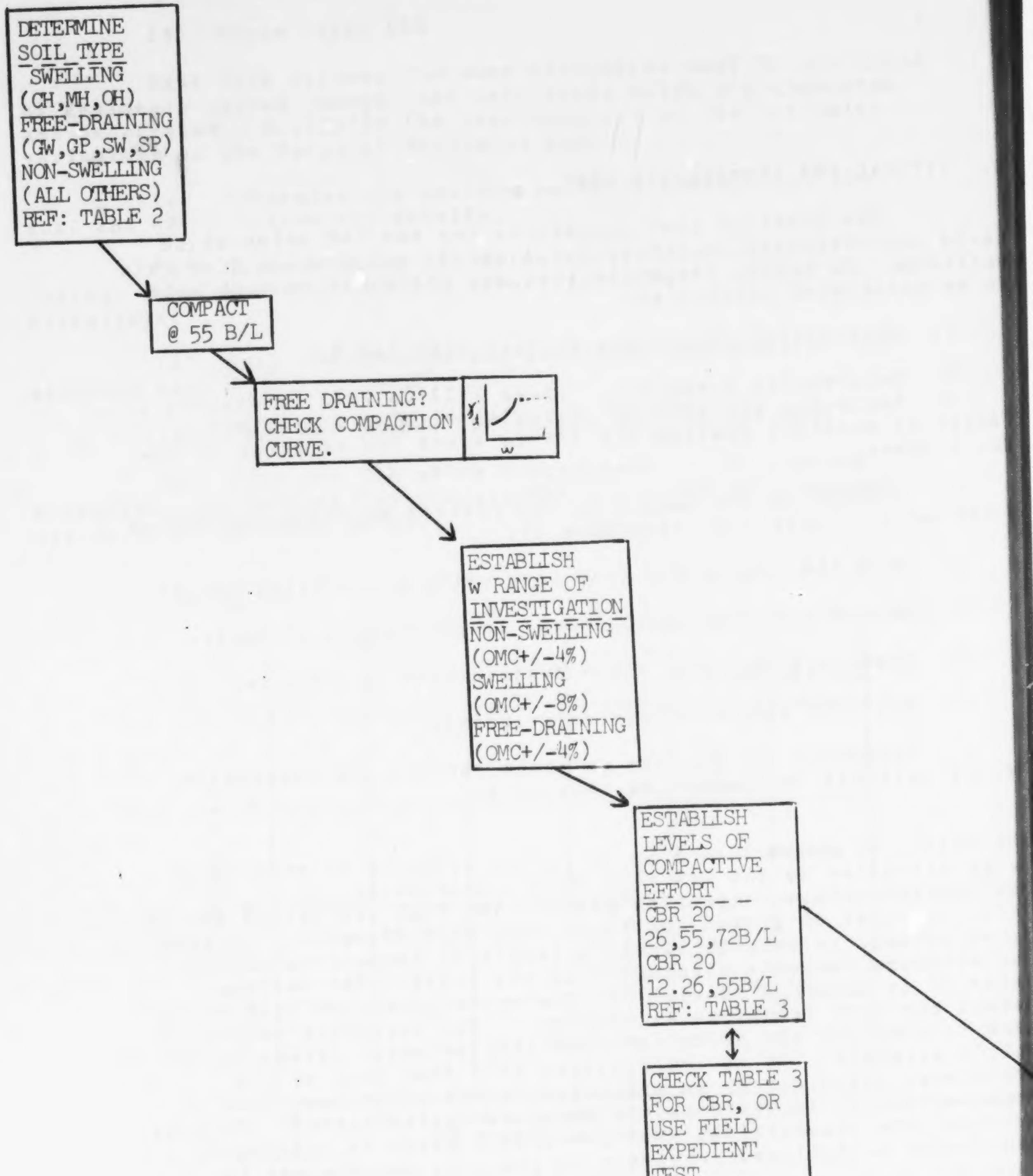
B. TYPICAL CBR PENETRATION TEST

The detailed test procedures for the CBR penetration tests are contained in references 1 and 2, and Section X of this workbook. In brief, the steps that are performed on each mold to be penetrated consist of:

1. Oven-drying the sample at less than 140 F.
2. Determining gradation. (para 2-23 to 2-27 references 1)
3. Replacing all +3/4-in. material with an equivalent weight of material passing the 3/4-in sieve but retatine on the No. 4 sieve.
4. Compacting the sample at the desired moisture content in a CBR mold. (para 2-81 reference 1).
5. Soak the sample for four days using a surcharge weight.
6. Compute the free swell index, or percentage of swell.
7. Penetrate the mold after draining for 15 minutes.
8. Plot the stress-strain relationship.
9. Determine CBR for the moisture content and compactive effort utilized in compacting this mold.

The object of steps 1 to 3 is to reduce bridging or wedging of large particles in the CBR mold which could cause non-representative compaction results and high CBR values due to relative sizes of piston, particle, and mold diameter. In step 4, an attempt is made to simulate levels of compactive effort and moisture contents obtainable in the field. The soaking (step 5) is intended to simulate the worst condition that could befall the soil (complete saturation). The surcharge weight is used to simulate the effect of overlying pavement layers on the soil's strength. It is important to note that many of the procedural steps are of an arbitrary nature which have been compensated for in the flexible pavement design curves. Because of this, the standardized test procedures given in references 1 and 2 must be followed precisely or the test results may be invalid.

CBR TESTING PROGRAM FLOW CHART


DETERMINE SOIL TYPE
SWELLING (CH,MH,OH)
FREE-DRAINING (GW,GP,SW,SP)
NON-SWELLING (ALL OTHERS)
REF: TABLE 2
COMPACT @ 55 B/L
FREE DRAINING? CHECK COMPACTION CURVE.
γd
w
ESTABLISH w RANGE OF INVESTIGATION
NON-SWELLING (OMC+/-4%)
SWELLING (OMC+/-8%)
FREE-DRAINING (OMC+/-4%)
ESTABLISH LEVELS OF COMPACTIVE EFFORT
CBR 20
26,55,72B/L
CBR 20
12.26,55B/L
REF: TABLE 3
CHECK TABLE 3 FOR CBR, OR USE FIELD EXPEDIENT TEST

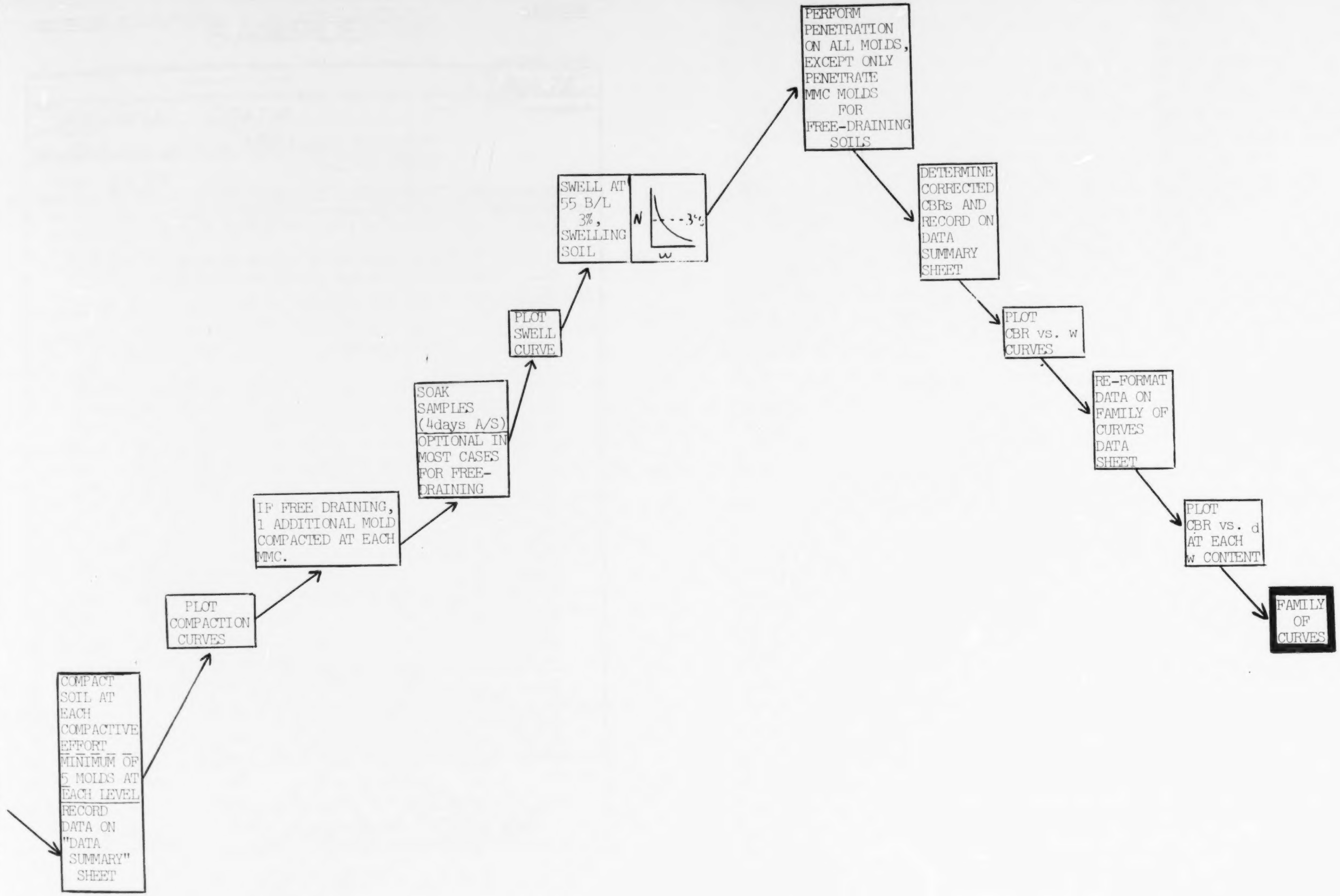

COMPACT SOIL AT EACH COMPACTIVE EFFORT
MINIMUM OF 5 MOLDS AT EACH LEVEL
RECORD DATA ON "DATA SUMMARY" SHEET
PLOT COMPACTION CURVES
IF FREE DRAINING, 1 ADDITIONAL MOLD COMPACTED AT EACH MMC.
SOAK SAMPLES (4days A/S)
OPTIONAL IN MOST CASES FOR FREE-DRAINING
PLOT SWELL CURVE
SWELL AT 55 B/L 3%, SWELLING SOIL
N
3%
w
PERFORM PENETRATION ON ALL MOLDS, EXCEPT ONLY PENETRATE MMC MOLDS FOR FREE-DRAINING SOILS
DETERMINE CORRECTED CBRs AND RECORD ON DATA SUMMARY SHEET
PLOT CBR vs. w CURVES
RE-FORMAT DATA ON FAMILY OF CURVES DATA SHEET
PLOT CBR vs. d AT EACH w CONTENT
FAMILY OF CURVES

TABLE 1. SUMMARY OF COMPACTION REQUIREMENTS

MATERIAL	PERCENTAGE COMPACTION OF MATERIALS WITH DESIGN CBR VALUES OF 20 AND ABOVE
Base Course	No less than 100% of CE 55 maximum density
Subbase and Subgrade	No less than 100% of CE 55 maximum density
	PERCENTAGE COMPACTION OF MATERIALS WITH DESIGN CBR VALUES BELOW 20
Select material and subgrade in fills	Cohesionless fill will not be placed at less than 95% of CE 55 maximum density. Cohesive fill will not be placed at less than 90% of CE 55 maximum density.

NOTE: A cohesive soil is one with a PI above 5.
A cohesionless soil is one with a PI ≤ 5.

TABLE 2. SUMMARY OF CBR TEST PROGRAMS

TEST PROGRAM	TYPE SOIL NORMALLY TESTED*	COMPACTION BLOWS/LAYER	PROBABLE USE OF TEST RESULTS
	REMOLDED LABORATORY TESTS		
Non-swelling soils	All except CH, MH, OH, GW, GP, SW, & SP	For CBR ≥ 20 26, 55, 72 For CBR < 20 12, 26, 55	For compacted subgrade, select, and subbase mat.
Swelling soils	CH, MH, & OH	12, 26, 55	Low quality compacted subgrades
Free draining soils	GW GP, SW, & SP	26, 55, 72	Compacted subgrade, select, & subbase mat.
	TESTS FOR INSITU STRENGTHS		
Field CBR	All	N/A	Uncompacted subgrade
Undisturbed sample	CH, MH & OH	N/A	Uncompacted subgrade - allows evaluation of swell

---

*This categorization is intended to serve as a guide for planning laboratory activities. Deviations may be noted in the initial stages of a test program which will dictate adjustments.

SAMPLE

SWELL DATA CURVE

DATE 7 AUG 75

PROJECT AIRFIELD DELTA

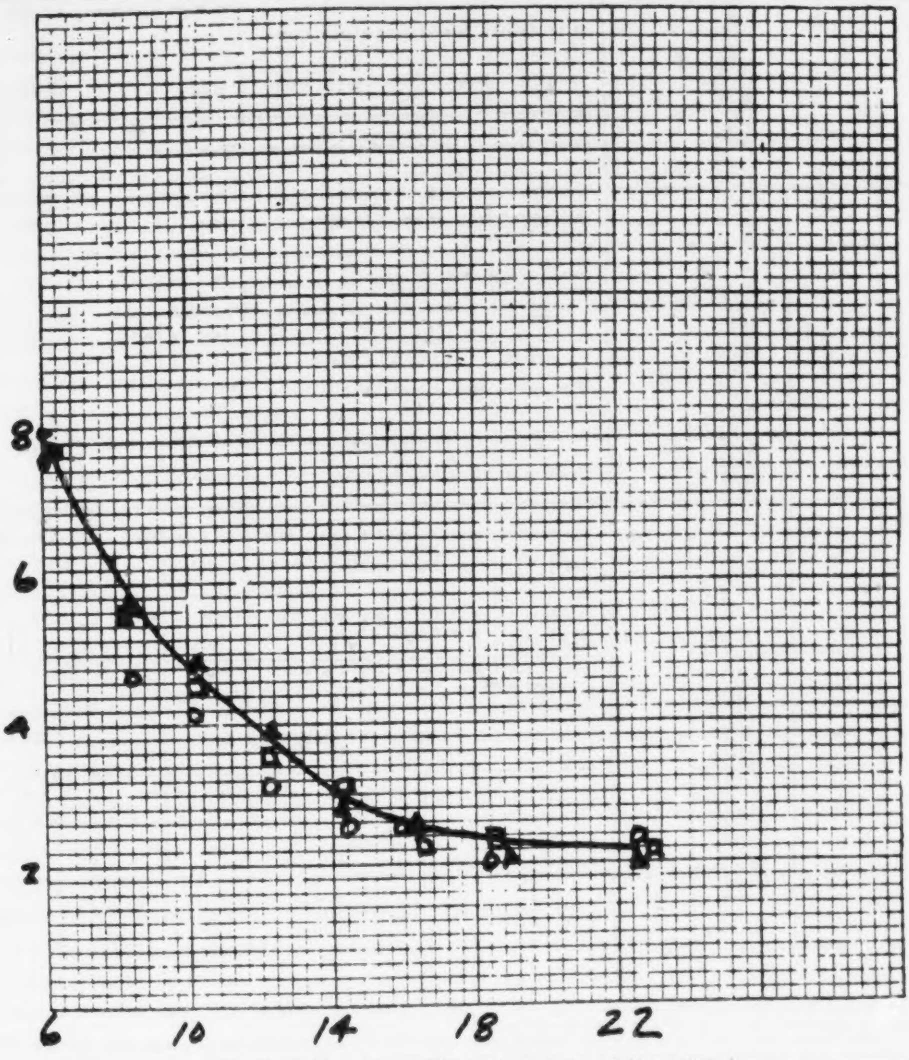
EXCAVATION NUMBER 10

SAMPLE NUMBER 10-1

REMARKS (Use reverse side, if more space is needed)

STA 8+50

RESULTS



TECHNICIAN (Signature) SP5 HARDCORE

PLOTTED BY (Signature) PFC AUGERHEAD

CHECKED BY (Signature) Sandy Hill

TABLE 3. CHARACTERISTICS PERTINENT TO ROADS AND AIRFIELDS

CORPS OF ENGINEERS

Major Divisions (1)	(2)	Symbol Letter (3)		Symbol Hatching (4)	Symbol Color (5)	Name (6)	Value as Subgrade When Not Subject to Frost Action (7)	Value as Subbase When Not Subject to Frost Action (8)
COARSE-GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		○	Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent
		GP		●		Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good
		GM	d	▦	Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good
			u				Good	Fair
		GC		▨		Clayey gravels, gravel-sand-clay mixtures	Good	Fair
	SAND AND SANDY SOILS	SW		⁘	Red	Well-graded sands or gravelly sands, little or no fines	Good	Fair to good
		SP		⁘		Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair
		SM	d	⁞	Yellow	Silty sands, sand-silt mixtures	Fair to good	Fair to good
			u				Fair	Poor to fair
		SC		▨		Clayey sands, sand-clay mixtures	Poor to fair	Poor
FINE-GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 5	ML		▥	Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable
		CL		▨		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable
		OL		▥		Organic silts and organic silt-clays of low plasticity	Poor	Not suitable
	SILTS AND CLAYS LL IS 50 or greater	MH		▥	Blue	Inorganic [illegible] ticaceous or diatomaceous fine sandy [illegible] lty soils, elastic silts	Poor	Not suitable
		CH		▨		Inorganic clays [illegible] high plasticity, fat clays	Poor to fair	Not suitable
		OH		▨		Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable
HIGHLY ORGANIC SOILS		Pt		≋	Orange	Peat and other highly organic soils	Not suitable	Not suitable

Note

1. Column 3, division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is on basis of Atterberg plasticity index is 5 or less, the suffix u will be used otherwise.
2. In column ..., the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and are listed because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination
   a. Processed base materials and other angular materials. Steel-wheeled and rubber-tired rollers are recommended for hard, angular materials subject to degradation.
   b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. Equipment size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:
      Crawler-type tractor -- total weight in excess of 30,000 lb.
      Rubber-tired equipment -- wheel load in excess of 15,000 lb, wheel loads as high as 40,000 lb may be necessary to obtain the required
      Sheepsfoot roller -- unit pressure (on 6- to 12-sq-in. foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be
        be at least 5 per cent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
3. Column 14, unit dry weights are for compacted soil at optimum moisture content for modified AASHO compaction effort.
4. In column 15, the maximum value that can be used in design of airfields is, in some cases, limited by gradation and plasticity requirements.

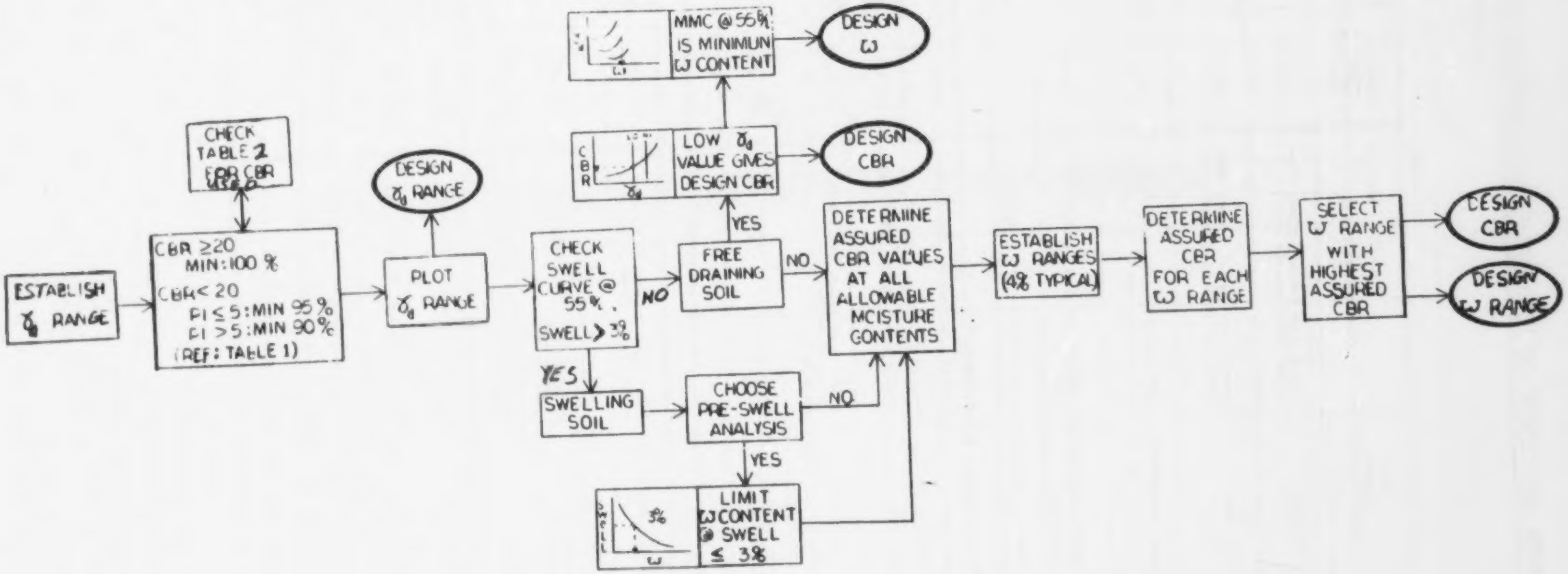
Value as Base When Not Subject to Frost Action (9)	Potential Frost Action (10)	Compressibility and Expansion (11)	Drainage Characteristics (12)	Compaction Equipment (13)	Unit Dry Weight lb per cu ft (14)	Typical Design Values	
						CBR (15)	Subgrade Modulus k lb per cu in. (16)
Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125-140	40-80	300-500
Fair to good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110-140	30-60	300-500
Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125-145	40-60	300-500
Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	115-135	20-30	200-500
Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	130-145	20-40	200-500
Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	110-130	20-40	200-400
Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	105-135	10-40	150-400
Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120-135	15-40	150-400
Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-130	10-20	100-300
Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-135	5-20	100-300
Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90-130	15 or less	100-200
Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90-130	15 or less	50-150
Not suitable	Medium to high	Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90-105	5 or less	50-100
Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller, rubber-tired roller	80-105	10 or less	50-100
Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	90-115	15 or less	50-150
Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	80-110	5 or less	25-100
Not suitable	Slight	Very high	Fair to poor	Compaction not practical	-	-	-

limits, suffix d (e.g., GMd) will be used when the liquid limit is 25 or less and the

thickness of lift are properly controlled. In some instances, several types of equipment
of two types may be necessary.
with limited fines or screenings. Rubber-tired equipment is recommended for softer materials

ensities for some materials (based on contact pressure of approximately 65 to 150 psi).
cessary to obtain the required densities for some materials. The area of the feet should

CBR FAMILY OF CURVES ANALYSIS


ESTABLISH γd RANGE
CBR ≥20 MIN:100 %
CBR< 20 PI ≤ 5:MIN 95 % PI > 5:MIN 90 %
(REF: TABLE 1)
CHECK TABLE 2 FOR CBR
PLOT γd RANGE
DESIGN γd RANGE
CHECK SWELL CURVE @ 55% . SWELL > 3%
YES
NO
SWELLING SOIL
FREE DRAINING SOIL
YES
NO
LOW γd VALUE GIVES DESIGN CBR
DESIGN CBR
MMC @ 55% IS MINIMUN ω CONTENT
DESIGN ω
CHOOSE PRE-SWELL ANALYSIS
NO
YES
LIMIT ω CONTENT @ SWELL ≤ 3%
3%
SWELL
ω
CBR
γd
DETERMINE ASSURED CBR VALUES AT ALL ALLOWABLE MOISTURE CONTENTS
ESTABLISH ω RANGES (4% TYPICAL)
DETERMINE ASSURED CBR FOR EACH ω RANGE
SELECT ω RANGE WITH HIGHEST ASSURED CBR
DESIGN CBR
DESIGN ω RANGE

V. TEST PROGRAM FOR NON-SWELLING SOILS

This test program is applicable to the majority of soils used in construction. As Table 2 indicates, soils which fall into this grouping might be used as compacted subgrade, select, or subbase materials depending upon their strengths and location in regard to the construction site. To illustrate the methodology of evaluating the design CBR, the data given on the CBR analysis sheet for "Hope Road" will be used. This data was taken from the subgrade along a proposed road alignment. The object of the following analysis is to determine a soil placement moisture content range for a specified level of compactive effort which gives the greatest assured design CBR. To develop this data, the soils analyst must accomplish the following:

(1) Establish the OMC of the soil at 55 blows/layer. This is done using the standard CE 55 compaction test. For this example OMC = 14%.

(2) Establish a moisture range for CBR investigation. Reference 2 states that the range is OMC ± 4%. This is a time saving guide as experience has shown that the maximum CBR will normally occur at compaction moisture contents within this range and that testing soils beyond these limits is wasted effort. For this example, the moisture content range of investigation is 10 to 18%.

(3) Compact samples within the moisture content range of investigation at different levels of compactive effort. This is done to allow for evaluation of soil strength when field placement is at something other than 100% CE 55. The levels of compactive effort to be selected in the laboratory are dependent upon the amount of compactive effort stipulated by the compaction requirements (Table 1). If the soil is expected to be used as compacted subgrade or select material, it will be placed at less than 100% CE 55, and laboratory compactive efforts of 12, 26, and 55 blows per layer are usually selected. If the soil is a very high quality subgrade (CBR >20) or a subbase, the laboratory tests should include samples compacted in excess of 55 blows per layer. Normally 26, 55, and 72 blows per layer are adequate. The only stipulation placed on the levels of compactive effort to be used is that a 55 blow per layer compaction curve be obtained, and that data is developed at two other levels of compactive effort encompassing the specified field placement densities. In this example, the soil is a CL (cohesive) subgrade so samples were compacted at 12, 26, and 55 blows per layer.

(4) Soak samples and measure swell. The initial assumption from Table 2 was that this is a non-swelling soil. It is emphasized that this categorization based upon USCS classification is only an approximation. The swell must be monitored to verify the fact that the soil being tested is in fact non-swelling.

(5) Perform CBR penetration tests and determine corrected CBR for each sample. This is done according to the technique previously discussed in para IV. Important is the fact that accumulation of the required data involves a considerable amount of work. At a minimum, 15 molds (5 per level of compactive effort) must be made. In this example 21 molds were compacted, soaked, then penetrated. The results are given in the "Data Summary."

(6) Plot the data on graphs of dry density vs. molding moisture content and corrected CBR vs. molding moisture content.

(7) Reformat data on CBR family of curves data sheet.

This step is performed in order to identify values of dry density and CBR for whole integer moisture contents within the range of investigation at each level of compactive effort. Some of this data may be transferred directly from the data summary table. The remainder must be interpolated from the CBR/dry density vs. molding moisture content graphs.

(8) Plot the CBR family of curves. The object of this step is to place the laboratory data in a form which lends itself to analysis. In essence, what is being done is the establishment of trends of strength variation as the moisture content and dry density change. The blank places between actual data points are being filled in. Drawing a CBR family of curves involves considerable practice, numerous attempts, and subsequent adjustments. From the example, it can be seen the available data has been plotted on a graph of corrected CBR vs. molded dry density for constant moisture contents. Here it can be seen that for low molding water contents (i.e. from 10 to 12%) there was an increase in strength with dry density, and at high moisture contents (i.e. 16, 17, and 18%), the reverse was found to be true. For the intermediate moisture contents, there was an increase in strength to some point, and then a decrease as the dry density increased.

ENGINEERING ANALYSIS - The work done to this point is the job of the laboratory technician. However, it is the responsibility of the engineer to insure that the CBR data is properly obtained and presented. More importantly, analysis of the data with an understanding of how the result affects the design and impacts on economic considerations is the key responsibility of the engineer. Prior to considering the details of the analysis, it is possible to observe two points of interest about the soil being used for the example. First, the maximum strength of this soil does not occur at OMC but at 1.4% dry of that value (see corrected CBR vs. molding water content). Secondly, from the CBR family of curves, it appears that for some moisture contents, the soil looses strength as the dry density or level of compactive effort increases. The impact of these two factors is important in that if maximum strength from this soil is desired, that strength might be achieved at a moisture content dry of OMC and at some level of compactive effort less than CE 55. The reasons for these two phenomena are highly speculative. For purposes of this course, it is essential that the engineer realize that soil placement at OMC and the most compactive effort possible are not always the answers to good construction.

(9) Establish a density range at which soil will be placed in the field. Minimum limits for field compaction requirements are based on settlement criteria and are given in Table 1. To facilitate construction, it is normal practice to specify a reasonable range of densities which can be economically obtained and then examine the strength values which would occur within that range. Establishment of the density range depends to a large degree on economics. The greater the latitude given the builder, the better are the chances of the soil being placed within established limits. However, if an extreme range is stipulated, the CBR value allowed for design might be reduced and thicker pavement structure could be required. Experience has shown that a five per cent range (i.e. 90 to 95% of max. CE 55 dry density) is reasonable. Another factor to consider is actual field compaction experience obtained from either a test strip or prior construction. Such data can be accumulated by measuring the in-place dry densities obtained for different numbers of passes with the available compaction equipment and determining the point where additional passes give little increase in density.

For this example, the soil according to Table 1 must be placed at at least 90% of max. CE 55 dry density. It will be assumed that experience with the soil shows that 93 to 98% of max. CE 55 dry density can be achieved with reasonable effort. This means that the limits of densities will be:

0.93 x 130 =120.9 pcf

0.98 x 130 = 127.4 pcf

These limits can be imposed on the "CBR family of curves" by drawing two vertical lines at 120.9 and 127.4 pcf.

10. Determine the assured CBR values between the specified density limits. If the builder is going to be allowed to place the soil between 120.9 and 127.4 pcf, this step involves determining the CBR values obtained for each moisture content. By observing the change in CBR for any specific moisture content line between the two density limits, it can be seen that a range of strengths is possible. Since the builder will be allowed to place the soil anywhere between the established density limits, the CBR value selected as a potential design strength should represent the worst case. Using 10% moisture as an example, the CBR at 120.9 pcf is 8.5. At 127.4 pcf the CBR is a maximum of 11.3. Of the two, the 8.5 is the minimum strength that can be counted on for the specified density range. This procedure was followed for the remaining moisture contents, and the results are tabulated as follows:

w(moisture)	CBR
10	8.5
11	9.2
12	10.0
13	11.3
14	13.6
15	10.3
16	7.4
17	5.4
18	4.2

11. Determine the CBR values for potential moisture content specification ranges. As with the density range specified in step 9, a moisture content range which can be economically achieved in the field is desired. Within the overall range of investigation (OMC ± 4%), a smaller specification range which gives the greatest assured CBR will be determined. Experience has shown that a four per cent range (i.e. some value ± 2%) is a reasonable requirement; however, this particular span is not intended to represent an absolute rule. The engineer may elect to specify a smaller range to achieve a larger design CBR and a reduction in pavement thickness requirements, but this savings in pavement materials may be offset by increased costs associated with difficulties in meeting the more stringent requirements. Conversely, for some soils an expanded moisture content specification may have little effect on the design CBR. For the example problem a four per cent specification ranges will be considered. One possible range is 10 to 14%. If the engineer specified that the soil be placed within these limits, the worst possible strength that would be realized is a CBR of 8.5 (see tabulation under step 10). Such an analysis can be done for the other possible four per cent ranges, and the result is as follows:

w(range)	CBR
10 - 14	8.5
11 - 15	9.2
12 - 16	7.4
13 - 17	5.4
14 - 18	4.2

12. Select the moisture content range which gives the greatest design CBR. In steps 10 and 11, CBR values were selected based upon the premise that the engineer is going to allow the builder to place the soil anywhere between potential moisture content and density specification limits. Now it is time for the engineer to select the set of limits he desires. From the tabulation in step 11, it can be seen that the soil if placed between 11 and 15 per cent moisture will give the largest of the possible CBR values. Thus 9.2 becomes the design CBR, and the field quality control measures to be used to insure that a CBR = 9.2 is achieved are:

a. Compaction from 93 to 98% of max. CE 55 dry density or 120.9 to 127.4 pcf.

Summary This technique for determining a design CBR provides to a strength measure of at least 9.2 when the associated density and moisture content ranges are adhered to. Of course greater strengths will be realized within the specified limits but the value obtained allows the engineer to size the structure for the worst condition. It should be noted that for this soil and the size of the limits used, the greatest assured strength did not occur for the four per cent moisture content range centered on OMC. This is a phenomenon which has been found to occur for many soils having an appreciable clay contents. Also of importance is the fact that the analysis was based upon an initial selection of density limits. The engineer may find it advantageous to evaluate other density limits that meet the minimums given in Table 1 to see if an adjustment to these limits yield greater strengths.

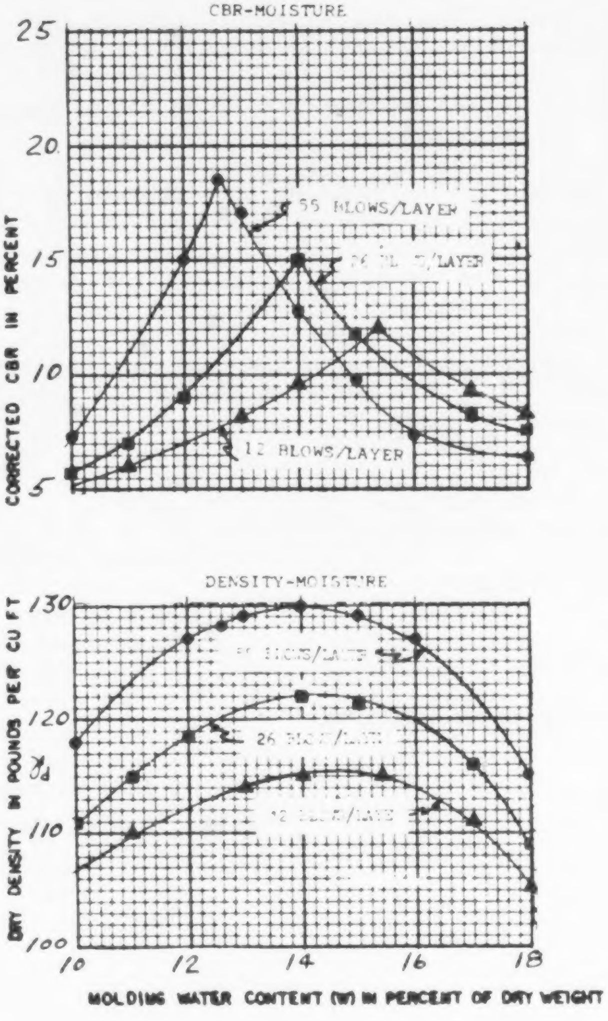
RECOMMENDED PROBLEMS:

CBR ANALYSIS SHEET

PROJECT HOPE ROAD
SOIL CLASSIFICATION CL
LOCATION OF SOIL STA 4+00
DATE OF TESTS 31 JUNE 1984
GROUP # N/A
NAME SP/4 WALKER, PFC DRILLOCK

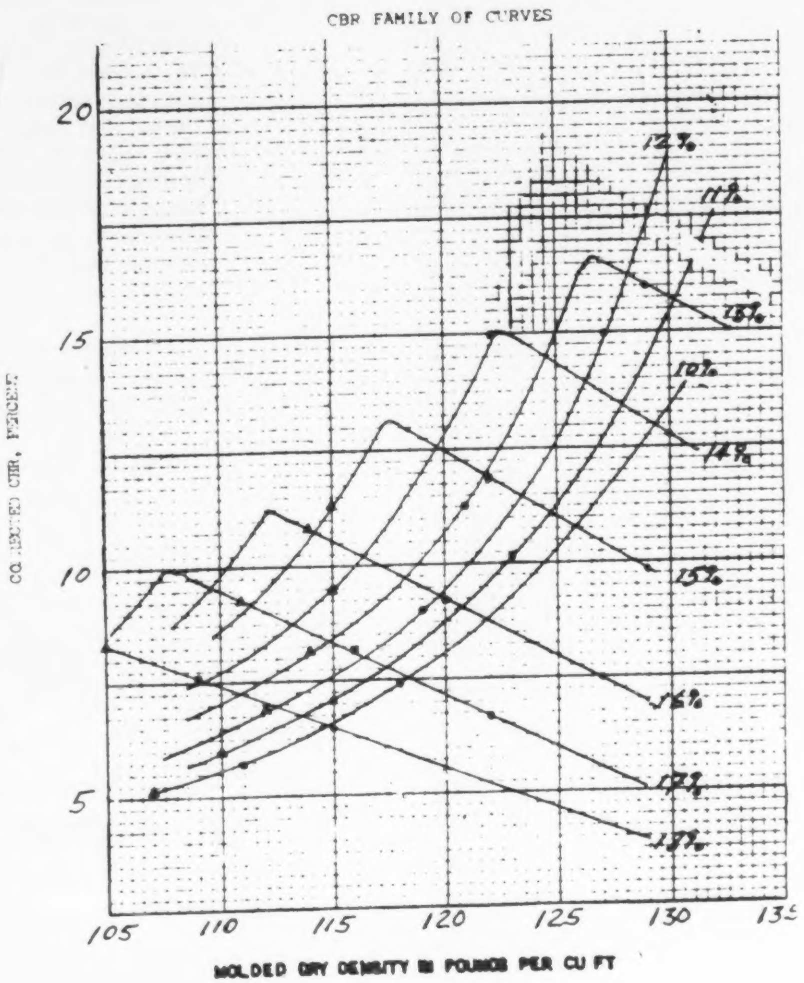
DATA SUMMARY

MOLD #	BLOWS PER LAYER	MOISTURE CONTENT w %	DRY DENSITY γ_d	CORRECTED CBR %
1	12	11	110	6.0
2		13	114	8.2
3		14	115	9.5
4		15.4	115	12.0
5		17	111	9.3
6		18	105	8.3
7	26	10	111	5.7
8		11	115	7.0
9		12	118.5	9.0
10		14	122	15.0
11		15	121.5	11.9
12		17	116	8.2
13		18	109	7.6
14	55	10	118	7.4
15		12	127	15.0
16		12.6	128	18.5
17		13	129	17.0
18		14	130	12.8
19		15	129	9.8
20		16	127	7.4
21		18	115	6.4



CBR FAMILY OF CURVES DATA SHEET

ω(%)		12 BLOWS/LAYER	26 BLOWS/LAYER	55 BLOWS/LAYER
10	γd	107 LBS/FT³	111 LBS/FT³	118 LBS/FT³
	CORR CBR	5.2 %	5.7 %	7.4 %
11	γd	110 LBS/FT³	115 LBS/FT³	123 LBS/FT³
	CORR CBR	6.0 %	7.0 %	10.2 %
12	γd	112 LBS/FT³	119 LBS/FT³	127 LBS/FT³
	CORR CBR	7.0 %	9.0 %	15.0 %
13	γd	114 LBS/FT³	121 LBS/FT³	129 LBS/FT³
	CORR CBR	8.2 %	11.5 %	17.0 %
14	γd	115 LBS/FT³	122 LBS/FT³	130 LBS/FT³
	CORR CBR	9.5 %	15.0 %	12.8 %
15	γd	115 LBS/FT³	122 LBS/FT³	129 LBS/FT³
	CORR CBR	11.3 %	11.8 %	9.8 %
16	γd	114 LBS/FT³	120 LBS/FT³	127 LBS/FT³
	CORR CBR	10.8 %	9.2 %	7.4 %
17	γd	111 LBS/FT³	116 LBS/FT³	122 LBS/FT³
	CORR CBR	9.3 %	8.2 %	6.7 %
18	γd	105 LBS/FT³	109 LBS/FT³	115 LBS/FT³
	CORR CBR	8.3 %	7.6 %	6.4 %



DESIGN CBR

COMPACTION RANGE: ______ % TO ______ %
______ LBS/FT³ TO ______ LBS/FT³

W%	LOWEST CBR	W% RANGE	ASSURED CBR
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	

DESIGN CBR (HIGHEST ASSURED) ______

DESIGN MOISTURE RANGE ______ % TO ______ %

VI. SWELLING OR EXPANSIVE SOILS

A. GENERAL

There is a small group of soils which can display the characteristics of objectionable volumetric expansion after being compacted and subjected to subsequent saturation. This presents a problem in pavement design as this expansion or swell can damage the structure by means of a "reverse-settlement" mechanism. The measure of swell used by the Corps of Engineers is termed the "free swell index" which is expressed as a percentage of the initial sample height. Objectionable swell is defined as that in excess of three per cent. Reference 6 explains the reason for the expansive nature of a soil as being primarily due to the type of clay minerals present. Montmorillonitic clay, which is just one of several kinds, appears to be comprised of the smallest and most highly charged particles found in nature. The combination of large surface area per unit volume and high surface charge causes a tremendous affinity for water and an equally tremendous ability to expand or shrink as water is taken in or removed from the soil.

Experience has shown that plasticity Index (PI) is an excellent indicator of expansive soils. The following guidelines have been suggested (Reference 8).

Swell Potential

PI	0-14	not expansive
PI	14-25	marginal
PI	25-40	critical
PI	40	highly critical

Although a high PI does not guarantee that the soil is expansive, critical soils should be checked more closely for swell tendencies.

B. ENGINEERING ALTERNATIVES

C. TEST PROGRAM

Table 2 indicates that the potentially expansive soils by USCS classification are the CH's, MH's, and OH's. The test procedure for determination of a design CBR for an expansive soil is similar to that previously discussed for the non-swelling soil, except that the objective is not exactly the same. For non-swelling soils, the object was to find the greatest assured CBR value for some range of densities and moisture contents; whereas, the object of the test program for the expansive soils is to find the moisture content ranges which will preclude objectionable swell and also provide the highest soaked CBR. Generally, the minimum swell and the highest soaked CBR will occur at a molding moisture content wet of OMC. The following is an example of such a test program for a CH soil taken from the subgrade at the proposed "Airfield Delta". As the majority of the steps are similar to those developed for the case of non-swelling soils, only differences will be discussed.

(1) Establish the OMC of the soil at 55 blows/layer.

(2) Establish a moisture range for CBR investigation. The OMC ± 4% range of investigation may not apply. It may be necessary to prepare samples over a wider range of moistures with the majority of the work being done on samples wet of OMC. In the example, samples were prepared over a range of OMC ±8%, and it will be illustrated later that much of the laboratory work done on samples dry of OMC is not essential.

(3) Compact samples within the moisture content range of investigation at different levels of compactive effort. As the type soil being tested will be cohesive and have a CBR less than 20, Table 1 indicates compaction must be at no less than 90% of max. CE 55 dry density. An upper limit can be established by reason of practicality as expansive soils are very difficult to compact at levels greater than 100% of max. CE 55 dry density. Therefore, laboratory compactive efforts of 12, 26, and 55 blows per layer are adequate for nearly all cases.

(4) Soak samples and measure swell. This step is a major deviation from the equivalent for the non-swelling case. For each sample, the expansion is measured, the percent swell is computed, and the percent swell is plotted against the molding water content. As an example, the triangular data point at 10 percent moisture on the attached "swell data curve" was obtained as follows:

Initial sample height = 4.50 in.
Height after soaking = 4.72 in.
Molding water content = 10.0%
Level of compactive effort = blows/layer

Free swell index

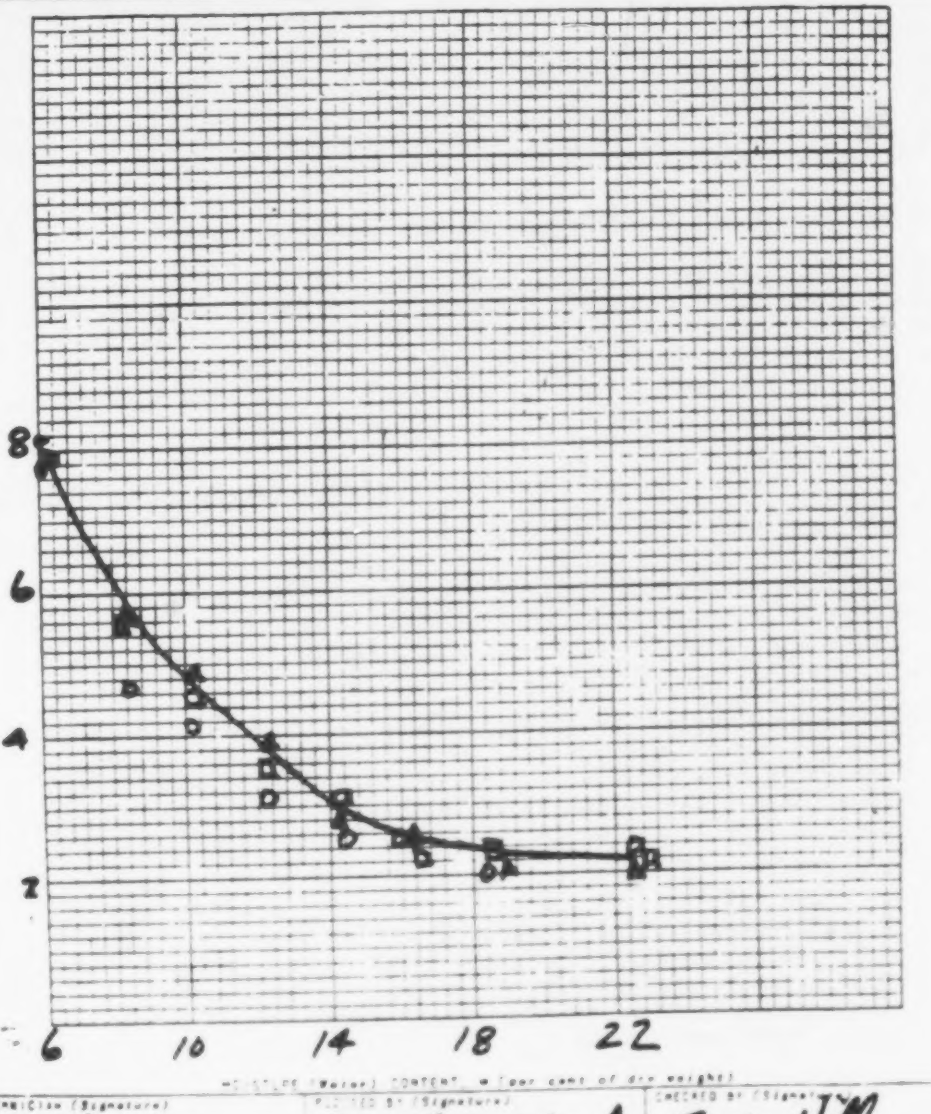
or

$$= \frac{4.72 - 4.50}{4.50} \times 100 = 4.9\%$$

Percentage of swell

After obtaining the swell index for each mold to be penetrated, these points are plotted on a swell vs. molding moisture content graph. A curve is then fit (usually to only the CE 55 data points), an example of which follows on the next page. From this particular curve, it can be ascertained that only soil placed and compacted at molding moisture contents of 14 percent or greater will swell three percent or less.

SWELL DATA CURVE	DATE 7 AUG 75
PROJECT AIRFIELD DELTA	RESULTS
EXCAVATION NUMBER 10	SAMPLE NUMBER 10-1
REMARKS (Use reverse side, if more space is needed) STA 8+50	



TECHNICIAN (Signature) 3AIS HARDCORE	PLOTTED BY (Signature) PFC HUGEEHEAD	CHECKED BY (Signature) Sandy Hill

(5.) Perform CBR penetration tests and determine corrected CBR for each sample.

(6.) Plot the data on graphs of dry density vs. molding moisture content and corrected CBR vs. molding moisture content.

(7.) Reformat data on CBR family of curves data sheet.

(8.) Plot the CBR family of curves.

(9.) Establish a density range at which soil will be placed in the field. For this example, it will be assumed that prior experience or benefit of a test strip is not available. Hence, only the guidelines from Table 1 will be used. The minimum level of compaction is 90% of max. CE 55 dry density and assuming a reasonable specification range, the upper limit will be set at 95% of max. CE 55 dry density. The actual densities are:

(110)(.90) = 99 pcf
(110)(.95) = 104.5 pcf

As in the previous example, the above limits are shown by two vertical lines on the CBR family of curves.

(10.) Determine the assured CBR values between the specified density limits. As more than 3 per cent swell is not acceptable, evaluation of the CBR values at moisture contents less than 14 per cent is needless. The CBR values for the applicable moisture contents are as follows:

w	CBR
14	3.4
15	4.2
16	3.2
17	2.2
18	1.3
19	.9
20	.4

(11.) Determine the CBR values for potential moisture content specification ranges.

w(range)	CBR
14 - 18	1.3
15 - 19	.9
16 - 20	.4

(12.) Select the moisture content range which gives the greatest design CBR. From step 11, it can be seen that the 14 to 18 per cent range gives largest CBR. Thus the design CBR is 1.3, and this value is obtained when the soil is placed at:

w = 14 - 18%

γ_d = 99.0 - 104.5 pcf

SUMMARY - The first thing the engineer must consider should be encounter an expansive soil is going some place else to build the structure. However, this is not always feasible, and this technique does allow for determination of a design CBR at which swell is not excessive. It is emphasized that obtaining this design value was done at the expense of strength. This technique does not provide for the advent of the soil being dryed to a moisture content less than the amount at placement. Should such extreme drying take place, excessive shrinkage and pavement failure might be expected. However, the engineer has to his advantage the fact that it takes considerable effort to remove water from expansive soils, and that such soils will normally be protected from drying by the overlying pavement. Another alternative to building on expansive soils is chemical stabilization. Fortunately, addition of small amounts of lime reduces considerably the potential for shrinkage and swell.

CBR ANALYSIS SHEET

PROJECT AirField DeLTA

SOIL CLASSIFICATION CH

LOCATION OF SOIL BoRRow PiT #5

DATE OF TESTS 7 Aug 75

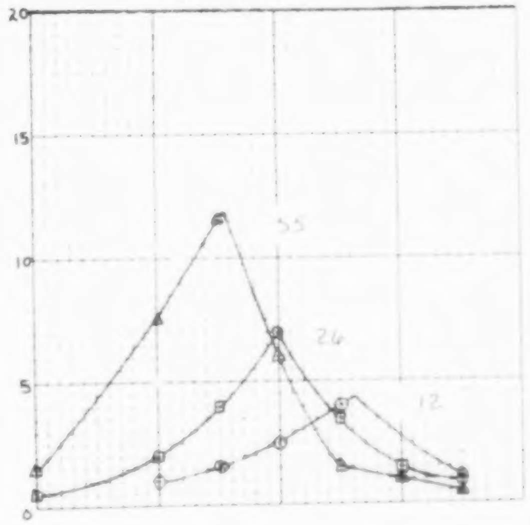
GROUP # NA

NAME SP5 HARDcoRe PFC AugeRhead

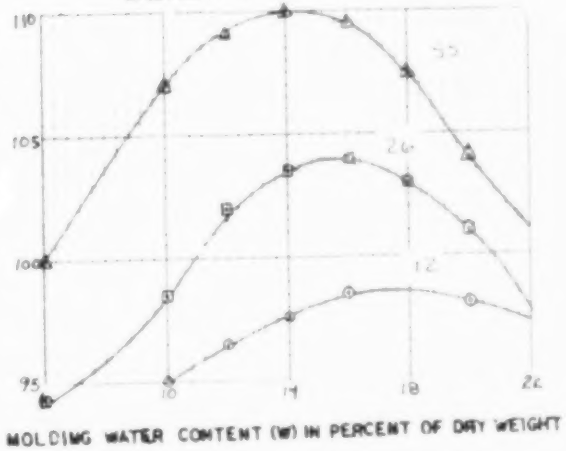
DATA SUMMARY

MOLD #	BLOWS PER LAYER	MOISTURE CONTENT ω %	DRY DENSITY γ_d	CORRECTED CBR %
1	12	10.0	95.0	1.0
2		12.0	96.5	1.5
3		14.0	97.5	2.5
4		16.0	98.5	4.0
5		20.0	98.0	1.2
6	26	6.0	94.0	0.5
7		10.0	98.5	2.0
8		12.0	102.0	4.0
9		14.0	103.5	7.0
10		16.0	104.0	3.5
11		18.0	103.0	1.5
12		20.0	101.0	1.0
13	55	6.0	100.0	1.5
14		10.0	107.0	7.5
15		12.0	109.0	11.5
16		14.0	110.0	6.0
17		16.0	109.5	1.5
18		18.0	107.5	1.0
19		20.0	104.0	0.5

CBR - MOISTURE



DENSITY - MOISTURE



CBR FAMILY OF CURVES DATA

ω(%)		12 BLOWS/LAYER	26 BLOWS/LAYER	55 BLOWS/LAYER
12	γ_d	96.5 LBS/FT^3	102.0 LBS/FT^3	109.3 LBS/FT^3
	CORR CBR	1.5 %	4.0 %	11.5 %
13	γ_d	97.1 LBS/FT^3	102.8 LBS/FT^3	109.7 LBS/FT^3
	CORR CBR	2.0 %	5.4 %	9.2 %
14	γ_d	97.6 LBS/FT^3	103.5 LBS/FT^3	110.0 LBS/FT^3
	CORR CBR	2.5 %	7.0 %	6.0 %
15	γ_d	98.1 LBS/FT^3	103.9 LBS/FT^3	109.8 LBS/FT^3
	CORR CBR	3.3 %	4.9 %	3.4 %
16	γ_d	98.4 LBS/FT^3	104.0 LBS/FT^3	109.5 LBS/FT^3
	CORR CBR	4.0 %	3.5 %	1.5 %
17	γ_d	98.6 LBS/FT^3	103.7 LBS/FT^3	108.8 LBS/FT^3
	CORR CBR	3.8 %	2.4 %	1.3 %
18	γ_d	98.5 LBS/FT^3	103.0 LBS/FT^3	107.5 LBS/FT^3
	CORR CBR	2.7 %	1.5 %	1.0 %
19	γ_d	98.4 LBS/FT^3	102.0 LBS/FT^3	105.8 LBS/FT^3
	CORR CBR	1.8 %	1.1 %	0.8 %
20	γ_d	98.0 LBS/FT^3	101.0 LBS/FT^3	104.0 LBS/FT^3
	CORR CBR	1.2 %	1.0 %	0.5 %

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ANALYSIS SHEET

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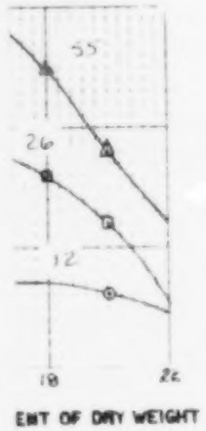
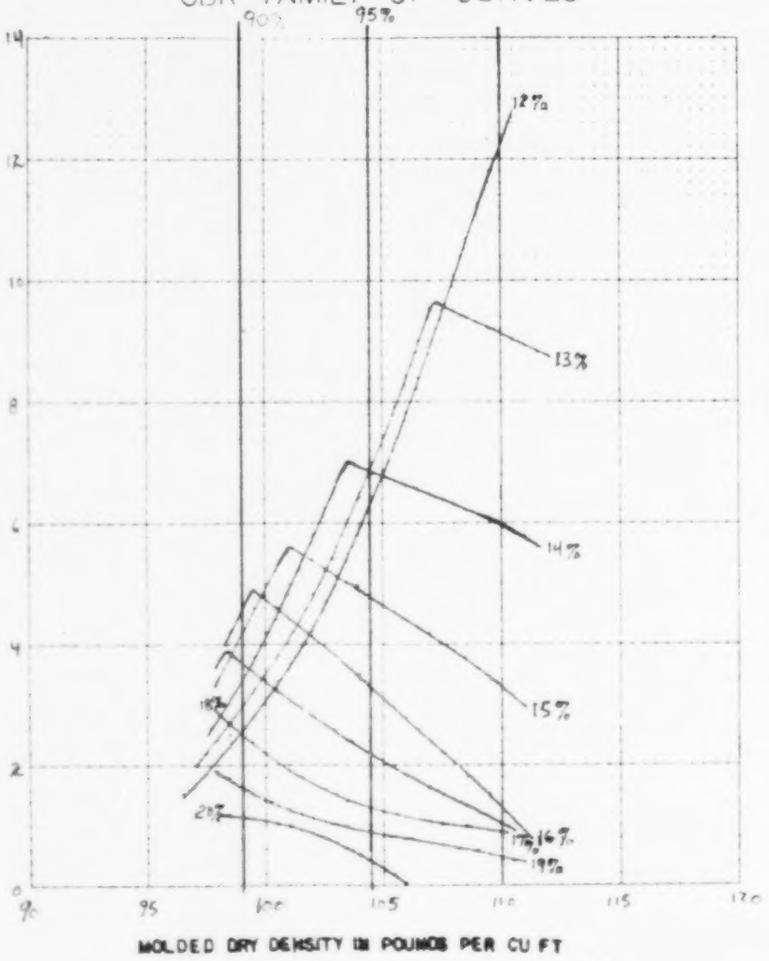


CBR FAMILY OF CURVES DATA

ω(%)		12 BLOWS/LAYER	26 BLOWS/LAYER	55 BLOWS/LAYER
12	γ_d	96.5 LBS/FT^3	102.0 LBS/FT^3	109.3 LBS/FT^3
	CORR C_{B_R}	1.5 %	4.0 %	11.5 %
13	γ_d	97.1 LBS/FT^3	102.8 LBS/FT^3	109.7 LBS/FT^3
	CORR C_{B_R}	2.0 %	5.4 %	9.2 %
14	γ_d	97.6 LBS/FT^3	103.5 LBS/FT^3	110.0 LBS/FT^3
	CORR C_{B_R}	2.5 %	7.0 %	6.0 %
15	γ_d	98.1 LBS/FT^3	103.9 LBS/FT^3	109.8 LBS/FT^3
	CORR C_{B_R}	3.3 %	4.9 %	3.4 %
16	γ_d	98.4 LBS/FT^3	104.0 LBS/FT^3	109.5 LBS/FT^3
	CORR C_{B_R}	4.0 %	3.5 %	1.5 %
17	γ_d	98.6 LBS/FT^3	103.7 LBS/FT^3	108.8 LBS/FT^3
	CORR C_{B_R}	3.8 %	2.4 %	1.3 %
18	γ_d	98.5 LBS/FT^3	103.0 LBS/FT^3	107.5 LBS/FT^3
	CORR C_{B_R}	2.7 %	1.5 %	1.0 %
19	γ_d	98.4 LBS/FT^3	102.0 LBS/FT^3	105.8 LBS/FT^3
	CORR C_{B_R}	1.8 %	1.1 %	0.8 %
20	γ_d	98.0 LBS/FT^3	101.0 LBS/FT^3	104.0 LBS/FT^3
	CORR C_{B_R}	1.2 %	1.0 %	0.5 %

CBR FAMILY OF CURVES



33

DESIGN CBR

COMPACTION RANGE: ______ % TO ______ %
______ LBS/FT^3 TO ______ LBS/FT^3

W%	LOWEST CBR	W% RANGE	ASSURED CBR
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	

DESIGN CBR
(HIGHEST ASSURED) ______

DESIGN MOISTURE
RANGE ______ % TO ______ %

TEST PROGRAM FOR FREE DRAINING SOILS

Determination of a design CBR for this group of soils requires the least testing of the three remolded laboratory test programs. Table 2 gives the USCS classification and the uses of the soils that fall in this group. The ease in testing is due to the free draining characteristics or lack of fines in the soil. From the CBR analysis sheet for Borrow Pit "A", it can be seen that the density vs. moisture content curves have a concaved upward shape and show maximum densities between seven and nine per cent moisture depending upon the level of compactive effort. For each curve, there is a limiting moisture content (shown by the triangle, diamond, or circle) at which moisture above that required to fill the voids after compaction is squeezed or drained from the soil. The dashed lines to the right of the limiting moisture contents represent attempts to compact the soil in a saturated condition but the result after compaction is densities and moisture contents at the limiting condition. This means that for such soils, field placement is relatively easy. To insure maximum dry density for any level of compactive effort, the only control measure that must be accomplished is to have more water available than that required for the maximum dry density at the appropriate level of compactive effort. From the corrected CBR vs. molding water content curves, it can be seen that the same pattern in relation to moisture content as does the dry density. Soils placed wet of the limiting moisture content achieve the maximum CBR possible for that level of compactive effort. In other words, moisture contents of loose soils above the limiting values have little bearing on the strength of a soil after compaction. This makes laboratory testing, field placement, and field control a relatively easy matter. To illustrate what must be done to arrive at a design CBR, the steps as outlined in the previous examples will again be followed.

(1) Establish the OMC of the soil at 55 blows/layer. The OMC for this example is the limiting moisture content or 8 per cent. To insure that a free draining soil is being tested, this curve should display a maximum dry density at a limiting moisture content.

(2) Compact samples at different levels of compactive effort. Compaction curves must be made for three levels of compactive effort up to the limiting moisture contents. As these soils frequently have CBR values greater than 20, the placement compactive effort is in excess of 100 per cent of max. CE 55 dry density. Therefore, 26, 55, and 72 blows/layer are usually used.

(3) Soak samples and measure swell. Swell measurements are not required and soaking can be eliminated when it is determined that saturation does not affect the strength.

(4) Perform CBR penetration tests. Only the samples at the limiting moisture contents for each level of compactive effort need be tested. Normally more than one sample at the limiting conditions will be made for each level of compactive effort and all should be tested.

(5) Plot the data on graphs of dry density vs. molding moisture content and corrected CBR vs. molding moisture content. Only the plot of dry density vs. molding water content is required. The corrected CBR vs. molding water content graph is presented only for discussion.

(6) Plot CBR family of curves. This graph can be condensed into a single line. The three data points are obtained by plotting the corrected CBR against the associated dry density at the limiting moisture content.

(7) Establish a density range at which the soil will be placed in the field. Using the criteria from Table 1 and noting that the CBR for this soil is always greater than 20, the minimum level of compaction allowed is 100 per cent of max. CE 55 dry density. As there is a lack of additional information, specify 100 to 105 per cent of max. CE 55 dry density. This means compaction between:

$$(1.00)(120) = 120 \text{ pcf}$$
$$(1.05)(120) = 126 \text{ pcf}$$

(8) Determine the design CBR and placement moisture content. From the family of curves, it can be seen that the minimum CBR is 70 at 120 pcf. The placement moisture content necessary to insure that this strength is obtained is 8 percent or greater.

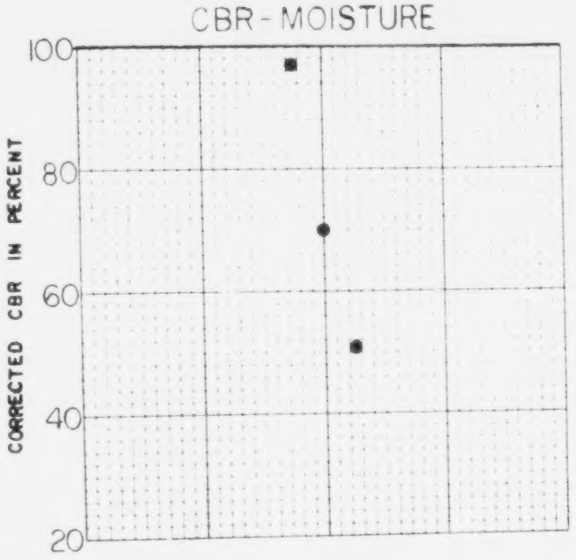
SUMMARY - It can be seen that considerably greater CBR values can be achieved if more field compaction is applied to the soil. If this is not too costly, it may be advantageous to specify greater densities. Lastly, it must be emphasized that the short cuts used in this procedure are dependent upon the fact that the soil is actually free draining. This can be determined from the shapes of the compaction curves.

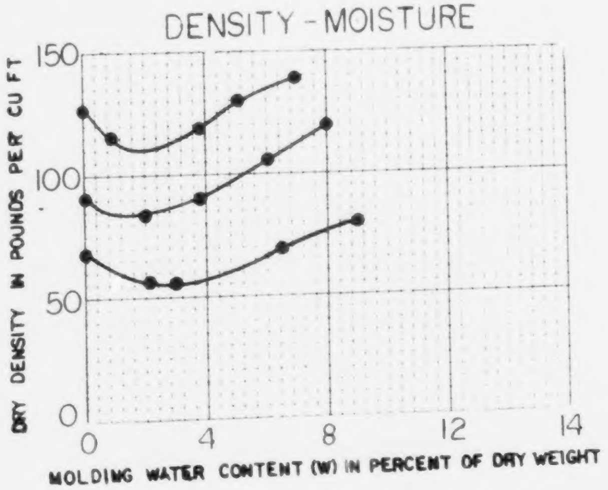
CBR ANALYSIS SHEET

PROJECT AIRFIELD BOLD EAGLE
SOIL CLASSIFICATION GW
LOCATION OF SOIL STA 3+50
DATE OF TESTS 1 MAY 84
GROUP #
NAME SP5 D.P. SPORKE

DATA SUMMARY

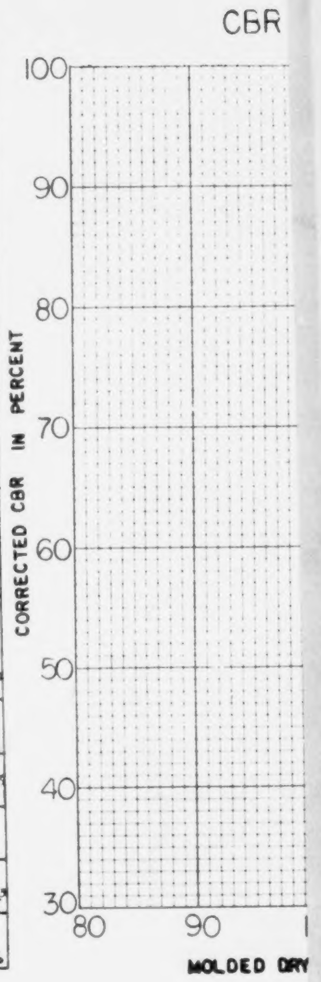
MOLD #	BLOWS PER LAYER	MOISTURE CONTENT ω %	DRY DENSITY γ_d	CORRECTED CBR %
1	26	0	68.0	
2	↓	2.1	56.3	
3		3.0	55.9	
4		6.5	69.1	
5		9.0	80.0	51
6	55	0	90.1	
7	↓	2.0	84.9	
8		3.8	90.0	
9		6.0	106.1	
10		8.0	120.0	70
11	72	0	126.0	
12	↓	1.9	115.1	
13		3.8	119.3	
14		5.1	131.9	
15		7.0	139.1	97





CBR FAMILY OF CURVES DATA.

ω(%)		BLOWS/LAYER	BLOWS/LAYER	BLOWS/LAYER
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%
	γ_d	LBS/FT^3	LBS/FT^3	LBS/FT^3
	CORR C_{BR}	%	%	%



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E

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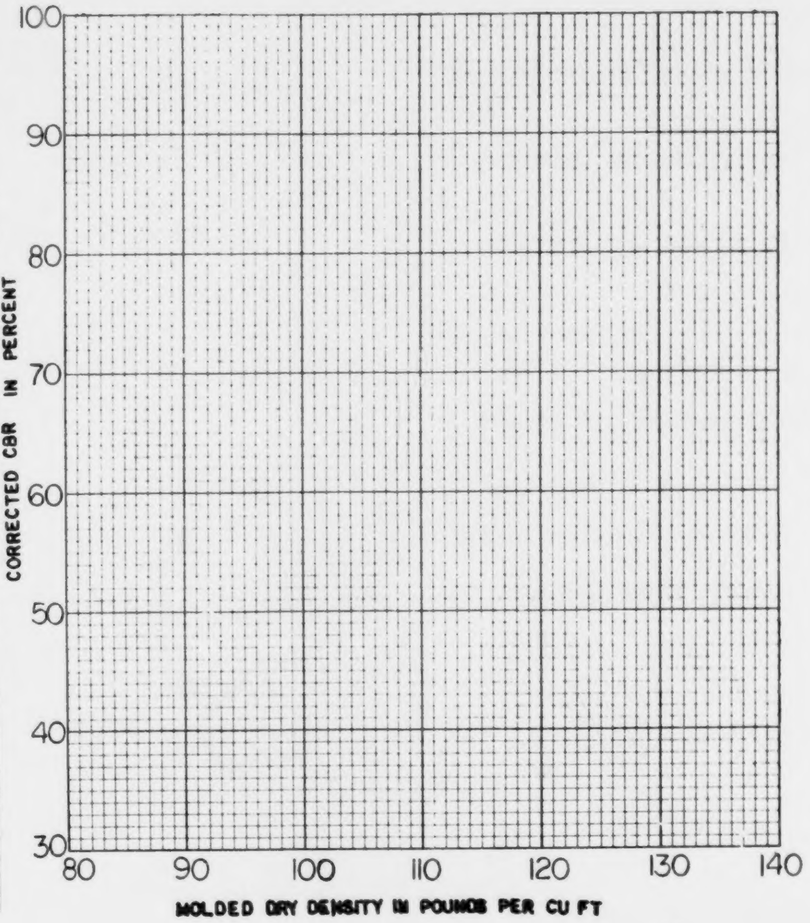
F DRY WEIGHT

ANALYSIS SHEET

CBR FAMILY OF CURVES DATA.

ω(%)		BLOWS/LAYER	BLOWS/LAYER	BLOWS/LAYER
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%
	γ_d	LBS/FT³	LBS/FT³	LBS/FT³
	CORR. C_{BR}	%	%	%

CBR FAMILY OF CURVES



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CBR ANALYSIS SHEET

DESIGN CBR

COMPACTION RANGE: ______ % TO ______ %
______ Lbs/Ft³ TO ______ Lbs/Ft³

W%	LOWEST CBR	W% RANGE	ASSURED CBR
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		DESIGN CBR (HIGHEST ASSURED) ______	
		DESIGN MOISTURE RANGE ______ % TO ______ %	

SOILS STABILIZATION IN THE THEATER OF OPERATIONS

I. Introduction

A. Objective: Using given soil test data, the student will be able to determine the most efficient stabilizing agent, the amount of stabilizing agent required, and the construction sequence required for the stabilizing agent.

B. Soil stabilization is defined as **any** process which improves the engineering properties of a soil.

II. Objectives of soil stabilization

A. Increase soil strength -

B. Reduce susceptibility to frost -

C. Control shrinking and swelling -

D. Improve workability -

E. Control the amount of moisture entering the soil (waterproofing) -

F. Dust control -

G. Allow for thinner pavements -

III. Basic design requirements for soil strength improvement

A. General requirements

1. Soil type -

2. Moisture content -

3. Compaction -

B. Thickness requirements

C. Laboratory testing

1. Lab tests are done to determine:

    a. the amount of stabilizer to be added.

    b. the amount of water to be added.

    c. the density to which the stabilized soil must be compacted.

    REFER TO **TM** 5-530 for specific test procedures.

2. Time required for tests.

    Because time is so precious in a tactical situation, the rapid completion of a project may over-ride the requirement for a complete test series.

TABLE 5-1. ESTIMATED TIME REQUIRED FOR TEST PROCEDURES

TYPE OF CONSTRUCTION	STABILIZING AGENT	TIME REQUIRED FOR TEST*
Expedient/Tactical	Lime	None
	Cement	None
	Bitumen	1 hour
Non-expedient/ deliberate	Lime	7 days
	Cement	6 to 9 days
	Bitumen	1 day

---

* These criteria do not include time required for gradation and classification tests on the untreated soil.

IV. Stabilization Methods and Selection

A. Methods

1. Mechanical stabilization

2. Chemical stabilization

    a. soil + lime -

    b. soil + cement -

    c. soil + bitumen -

3. Use of fabrics and membranes

B. Method selection process

1. Determine the soil gradation

2. Determine the plasticity index (PI)

a. lab method performed by soils analyst

b. field expedient method (SEE Table 5-2)

TABLE 5-2. Expedient PI identification

PLASTICITY DESCRIPTION	PLASTICITY INDEX RANGE	REACTION TO CRUSHING BETWEEN FINGERS
NON-PLASTIC	0 to 3	falls apart easily
SLIGHTLY PLASTIC	4 to 8	is easily crushed
MEDIUM PLASTIC	9 to 30	difficult to crush
HIGHLY PLASTIC	31 or more	impossible to crush

3. Identify the soil using the USCS or field ID card system.

4. Determine the type of stabilization as recommended in Figure 5-1.

V. Expedient Procedures

A. Mechanical Stabilization

1. The basic principle is to mix good soil with the bad soil to form acceptable soil. (SEE Appendix D, pp.25 to 28)

2. Know how to determine the soil type and its Atterberg Limits (PI).

3. Determine the gradation required.

4. Prepare the surface for stabilization (SEE Figure 5-2)

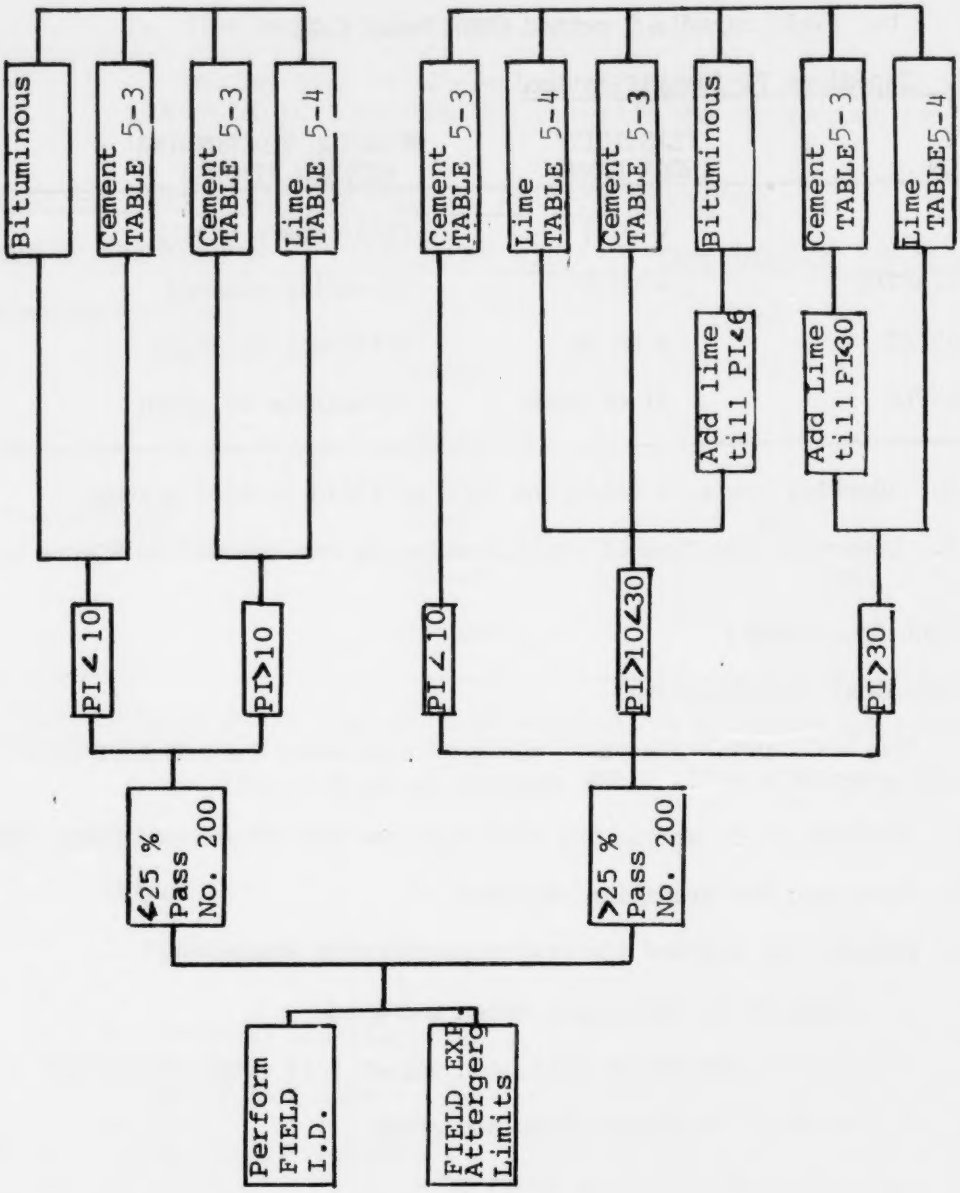
a. shape it to the proper crown and grade

b. scarify, pulverize soil, and adjust soil moisture content

c. re-shape to proper crown and grade

5. Add stabilizer evenly by means of -

FIGURE 5-1. SELECTION OF STABILIZING AGENT


Perform FIELD I.D.
FIELD EX Atterge Limits
≤25 % Pass No. 200
>25 % Pass No. 200
PI ≤ 10
PI>10
PI ≤ 10
PI>10≤30
PI>30
Bituminous
Cement TABLE 5-3
Cement TABLE 5-3
Lime TABLE 5-4
Cement TABLE 5-3
Lime TABLE 5-4
Cement TABLE 5-3
Bituminous
Add lime till PI≤6
Cement TABLE5-3
Add Lime till PI<30
Lime TABLE5-4

a. spreader(s)

b. spreader box(es) behind dump truck(s)

c. tailgating each truckload to cover a certain length

d. dumping in equally spaced piles, then grading into windrows and spreading evenly.

6. Mixing stabilizer with soil (SEE Figure 5-4)

a. For every 1% of portland cement add ½% water above the OMC and mix with a mixer, a pulvimixer, a blade, a scarifier, or a plow.

b. Mixing should continue until soil, aggregate, and stabilizer are mixed into a uniform, well-graded mass.

c. shape to the proper crown and grade.

7. Compaction (SEE Figure 5-5)

a. Begin compaction as soon as possible (within 4 hours) after adding water, to prevent water loss to evaporation.

b. Use sheeps-foot roller, pneumatic roller, or crawler-type tractor for initial compaction; finish with steel wheel roller or lightweight pneumatic roller.

FIGURE 5-5. COMPACTION OF STABILIZED MATERIAL

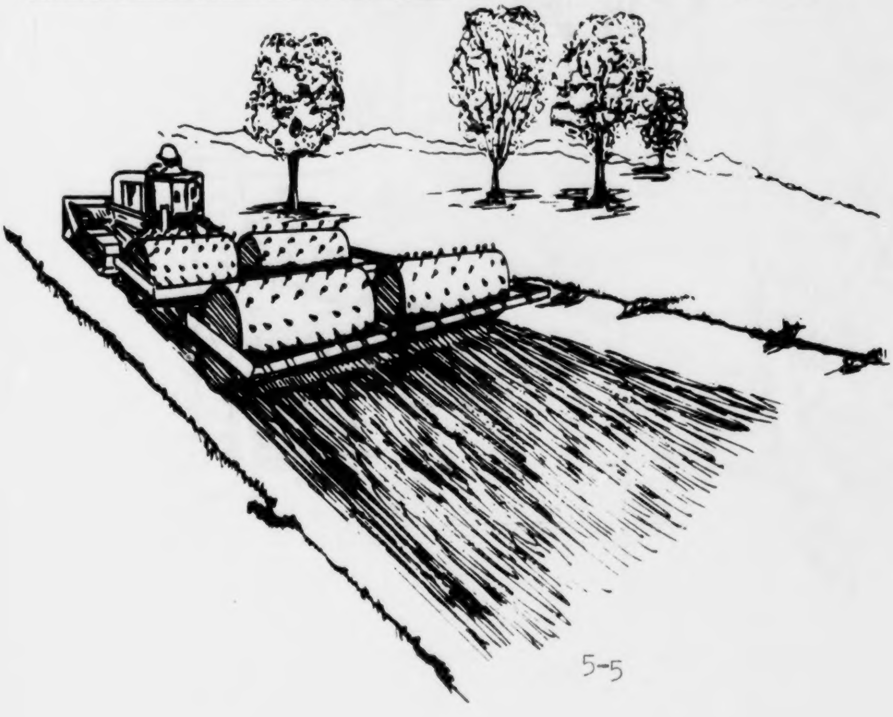


FIGURE 5-2. PREPARATION OF A SURFACE FOR IN-PLACE STABILIZATION

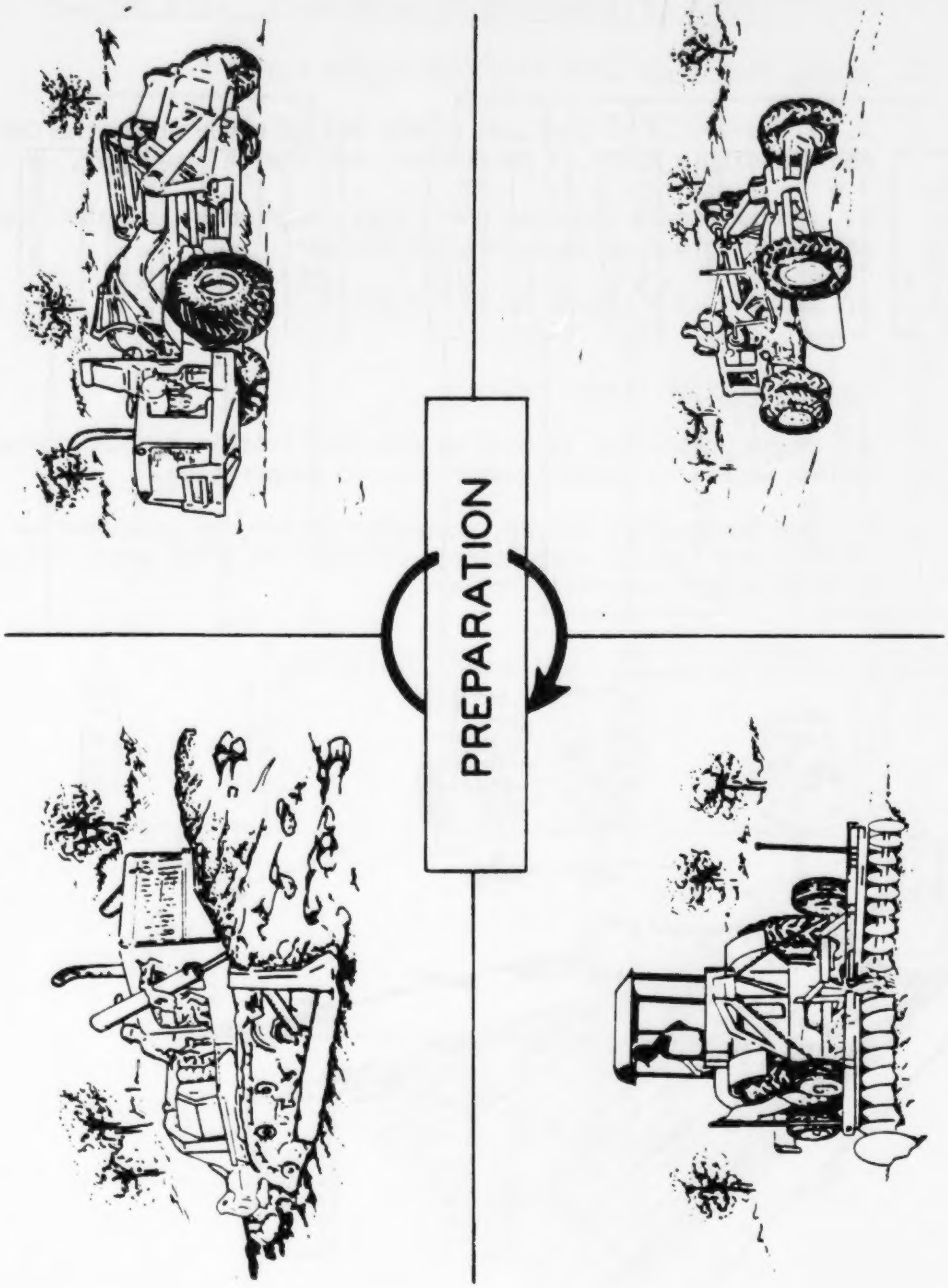

PREPARATION

FIGURE 5-3. SPREADING STABILIZER

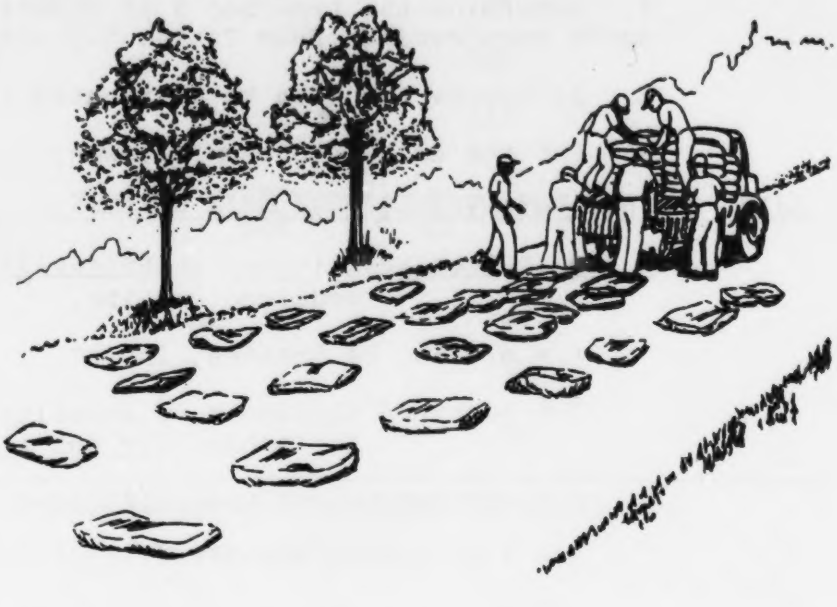
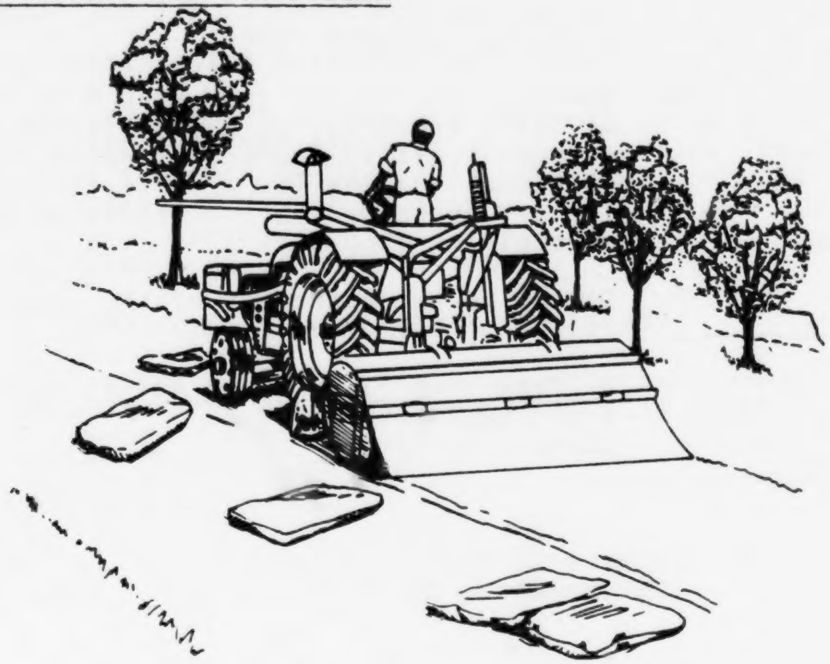


FIGURE 5-4. MIXING THE STABILIZER



B. Procedure for Estimating the Quantity of Stabilizing Agent Required.

1. Determine the soil type, Atterberg limits(PI), and dry density of the existing soil.

2. Determine the type and % of stabilizing agent requirements from Tables 5-3 and5-4.

3. Determine the area to be treated (in ft^2).

4. Put the data into the following formula:

$$Q = A \times \frac{T}{12} \times D \times \frac{S}{100}$$

Q = total quantity of stabilizing agent required, in lbs.

A = area to be treated, in ft^2.

T = required thickness of stabilized layer, in inches, for one lift only.

D = dry density of the existing soil, in $\frac{lbs}{ft^3}$.

S = % treatment specified.

5. Divide the quantity of stabilizer required(lbs) by the weight of each bag (94 lbs for cement or 100 lbs for lime) to get the number of bags required for each lift.

6. Multiply the answer from step 5 by the number of lifts required, to get the total number of bags required for the project.

7. (OPTIONAL) Allow for a waste factor of 10% for dry areas or 15% for wet areas by multiplying step 6 by 1.10 or 1.15.

8. Determine the placement of the bags by the following formula:

$$\frac{\text{A (from step 3)}}{\text{\# of bags/lift (from step 5)}} = X^2 = \text{\#of } ft^2 \text{ per bag}$$

$$\sqrt{X^2} = x = \text{\# of feet between bags}$$

C. Bituminous Stabilization (SEE Appendix D, pp. 18-24)

TABLE 5-3. ESTIMATED CEMENT REQUIREMENTS FOR VARIOUS SOILS*

Soil Classification	Estimated Cement %
GM, SM, GC, SC	8%
CL, ML, CH	10%
MH, OH	11%

TABLE 5-4. ESTIMATED LIME REQUIREMENTS FOR VARIOUS SOILS

Soil Classification	Estimated Lime %
GC	4%
SM, SC, GM	10%
CL, CH	8%

---

* for more precise design specifications, SEE Appendix D.

D. Fabric/membrane Stabilization

VI. Soil Stabilization Practical Exercises

1. You are ordered to analyze the soil in an area where a 4800' by 50' airstrip is to be built. You find that the soil is 0% gravel, 75% sand, and 25% clayey fines, with a PI of 8 or 9.

a. What type of soil is it?________

b. What type of stabilizer is needed?_________

c. How many bags of stabilizer will you need to stabilize the soil to a depth of 2 1/2 feet?

____

2. You are tasked to analyze the soil where a road, ten feet wide and 9000 feet long, is going to be built. You determine that the soil is GC, with 40% gravel, 35% sand, and 25% fines.

a. What type of stabilizer is needed?____________

b. How many bags of stabilizer will be needed to stabilize this roadway to a depth of 3 1/2 feet?

____

3. You analyze the soil at the site of a 70' by 5200' airstrip and find that the soil is SC, with 35% gravel, 40% sand, and 25% fines. It has a PI of 8 or 9.

a. What type of stabilizer is needed?__________

b. How many bags of stabilizer are needed to stabilize this soil to the depth of 3 1/2 feet?

____

SOIL TRAFFICABILITY

I. Soil trafficability is defined as the capability of a soil to support the cross-country movement of military vehicles.

II. Terminology

A. Cone Index(CI) - is an index of the shearing resistance of the soil, as obtained with the cone penetrometer. It is a dimension-less number representing the resistance to penetration into the soil of a 30° cone which has a ½sq.in. base area.

B. Critical Layer - is the layer in which the Rating Cone Index is considered a significant measure of trafficability, or the layer of soil which is regarded as being most pertinent to establishing the relationship between soil strength and vehicle performance. Its depth varies with the weight and type of vehicle and the soil profile, but it is normally the layer 6" to 12" below the surface.

C. Remolding Index - is the ratio of the remolded soil strength to the original soil strength, determined in accordance with the procedures described in the SOILS TRAFFICABILITY student workbook.

$$\text{Remolding Index (RI)} = \frac{\text{CI(after remolding)}}{\text{CI(before remolding)}}$$

D. Rating Cone Index (RCI) - is the measured cone index multiplied by the remolding index. It expresses the soil strength of an area.

$$RCI = CI \times RI$$

E. Vehicle Cone Index (VCI) - is the index assigned to a given vehicle that indicates the minimum soil strength, in terms of an RCI, that is required to permit a vehicle to pass over that soil a given number of times. VCI_1 is for 1 pass and VCI_{50} is for 50 passes. These VCIs are listed in Appendix C of this workbook.

III. Methods of Estimating Trafficability

A. Trafficability maps

KEY:

COLOR	LETTER SYMBOL	DRY PERIOD	WET PERIOD
dark green	A	passable	passable
light green	B	passable	doubtful to impassable
orange	C	doubtful	doubtful to impassable
red	D	impassable	impassable
red stripes	E	generally not passable due to steep slopes or rough ground. Soil not evaluated.	-----

B. Weather in an area.
  1. present conditions
  2. long-term conditions

C. Soil types

D. Estimated CI and RI (SEE Tables 6-1 and 6-2)

E. Topography of an area

F. Aerial photographs

G. Actually measuring the trafficability of the soil in place with a cone penetrometer, soil sampler, or remolding equipment.

TABLE 6-1. TRAFFICABILITY CHARACTERISTICS OF SOILS, IN WET SEASONS.

Group	Soils	Unified Soil Classification System	Probable Cone Index Range	Probable Remolding Index Range	Probable Rating Cone Index Range	Slipperiness Effects	Stickiness Effects	Comments
A	Coarse-grained, cohesionless sands and gravels	GW, GP, SW, SP	35 to 100*	Not applicable	Not applicable	Slight to none	None	Will support continuous traffic of tracked military vehicles or all-wheel-drive trucks with high flotation tires. Performance will increase with a decrease in tire pressure. Moist sands are good, dry sands only fair. Wheeled vehicles with standard tires may be immobilized in dry sands.
B	Inorganic clays of high plasticity, fat clays	CH	55 to 165	0.75 to 1.35	65 to 140	Severe to slight	Severe to slight	Usually will support more than 50 passes of military vehicles. Going will be difficult at times.
C	Clayey gravels, gravel-sand-clay mixtures Clayey sands, sand-clay mixtures Gravelly clays, sandy clays, inorganic clays of low to medium plasticity, lean clays, silty clays	GC SC CL	85 to 175	0.45 to 0.75	45 to 125	Severe to slight	Moderate to slight	Often will not support 40 to 50 passes of military vehicles, but usually will support limited traffic Going will be difficult in most cases.
D	Silty gravels, gravel-sand-silt mixtures Silty sands, sand-silt mixtures Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts Organic silts and organic silty clays of low plasticity Organic clays of medium to high plasticity, organic silts	GM SM ML and CL-ML MH OL OH	85 to 180	0.25 to 0.85	25 to 120	Moderate to slight	Slight	Usually will not support 40 to 50 passes of military vehicles. Often will not permit even a single pass. Going will be difficult in most cases.

TABLE 6-2. SOIL TRAFFICABILITY CLASSIFICATION IN USCS TERMS

High Topography, Wet-Season Condition

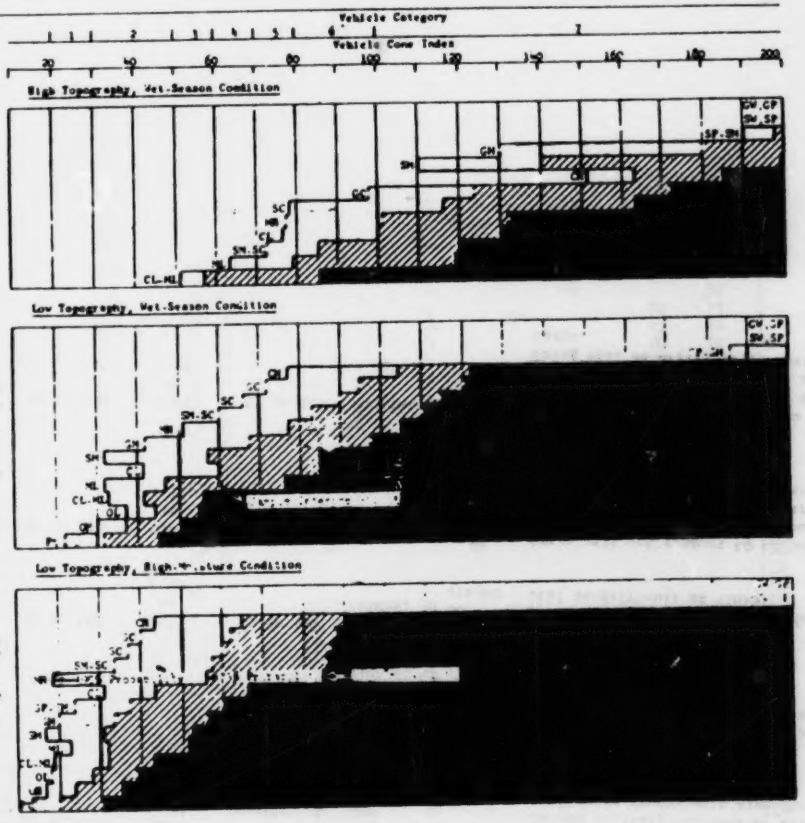
Soil Type Symbol	Strength Measurements Probable Range* CT	 RI	 RCI	 Mean RCI	Effects of Slipperiness**	 Stickiness
GW, GP SW, SP	...	...	...	...	None	None
SP-SM	125-241	1.19-2.17	196-316	256	None	None
GM	...	...	...	230†	None	None
SM	130-224	0.77-1.83	137-287	212	None	None
CH	167-217	0.84-1.10	158-210	184	Slight	Moderate
GC	...	...	...	165†	Slight	Slight
SC	127-231	0.72-0.98	104-208	156	Slight	Slight
MH	151-211††	0.32-1.00††	64-160††	112††	Slight	Slight
CL	123-211	0.59-0.95	82-180	131	Slight	Slight
SM-SC	167-185	0.47-1.13	65-211	138	Slight	None
ML	118-224	0.46-1.02	67-189	128	Slight	None
CL-ML	111-209	0.44-0.72	54-136	95	Slight	None

Low Topography, Wet-Season Condition

Soil Type Symbol	CT	RI	RCI	Mean RCI	Slipperiness**	Stickiness
GW, GP SW, SP	...	...	...	...	None	None
SP-SM	300	0.94	282	282†	None	None
CH	98-194	0.74-1.16	81-193	137	Severe	Severe
GC	...	...	...	130†	Moderate	Moderate
SC	77-257††	0.59-1.21††	61-255††	158††	Moderate	Moderate
SM-SC	160-216††	0.45-1.31††	72-208††	140††	Slight	None
MH	94-170	0.51-0.99	48-162	105	Severe	Moderate
GM	...	...	...	125†	Slight	None
SM	109-217	0.29-1.33	34-188	111	Slight	None
CL	90-188	0.46-0.88	46-146	96	Moderate	Moderate
ML	102-200	0.27-0.81	34-134	84	Moderate	Slight
CL-ML	85-165	0.31-0.69	34-96	65	Moderate	Slight
OL	95-135	0.38-0.74	41-89	65	Moderate	Slight
OH	64-164	0.32-0.78	14-110	62	Moderate	Slight
Pt	76-90††	0.65-0.57††	41-51††	46††	Moderate	Slight

Low Topography, High-Moisture Condition

Soil Type Symbol	CT	RI	RCI	Mean RCI	Slipperiness**	Stickiness
GW, GP SW, SP	...	...	...	...	None	None
CH	69-167	0.64-1.02	48-149	77	Severe	Severe
GC	...	...	...	70†	Severe	Moderate
SC	...	...	...	[illegible]†	Severe	Moderate
SM-SC	150-182††	0.45-0.63††	56-98††	52††	Moderate	Slight
MH	83-151	0.46-0.92	43-121	53	Severe	Severe
CL	71-155	0.[illegible]-0.50	13-117	[illegible]	Severe	Moderate
SP-SM	...	...	...	[illegible]†	Slight	None
GM	...	...	...	[illegible]†	Slight	Slight
SM	81-153	0.15-0.87	[illegible]-128	49	Slight	Slight
ML	91-171	0.26-0.60	[illegible]	45	Severe	Slight
CL-ML	71-155	0.27-0.53	[illegible]-96	46	Severe	Slight
OL	87††	0.56††	[illegible]††	[illegible]††	Moderate	Slight
OH	62-132	0.26-0.66	[illegible]-69	35	Severe	Slight
Pt	76-97††	0.45-0.61††	41-5[illegible]††	46††	Severe	Slight



PROBABILITY OF VEHICLE "GO" ON LEVEL TERRAIN

☐	Excellent	Greater than 90%
☐	Good	76 to 90%
▨	Fair	50 to 75%
■	Poor	Less than 50%

Good reliability, based on analysis of data

Fair reliability, based on judgement

Note: Vehicle category and cone index range are given in table 9-1.

* Based on +1 and -1 standard deviation from the mean.

** Applies only to wheeled vehicles without traction devices.

† Estimated from textural, plasticity, and organic properties of soil under given moisture condition

†† Based on analysis of less than five samples

‡ A vehicle with a vehicle cone index of 60 would have a 50-75% chance of going on an ML soil of low topography, wet-season condition

IV. Instruments used to measure trafficability.

A. Cone Penetrometer -

B. Hvorslev Sampler -

C. Cylinder and hammer -

V. Measuring Trafficability

A. Range of CI.

B. Number of measurements required.
(SEE pp. B-24 to B-26)

C. Other Factors.

1. Critical layer -
2. Surface slope -
3. Soil stickiness -
4. Soil slipperiness -
5. Vegetation present -
6. Other obstacles -

PRACTICAL EXERCISE

1. Identify test sites along planned route.

2. At each site find the critical layer(s) for 1 and 50 passes (SEE page B-27, Appendix B of this workbook).

3. Number of penetration readings required to get an average, based on the initial cone index (ICI).
   a. If ICI is greater than 200, take 1 or 2 readings within a 3 foot radius.
   b. If ICI is 150 to 200, take 3 or 4 readings in a 3 foot radius.
   c. If ICI is 10 to 150, take 15 readings per test site.
   d. If ICI is less than 10, the area is not suitable for trafficability.

4. Record penetration readings on a data sheet(Figure 6-1).

FIGURE 6-1. TRAFFICABILITY TEST DATA

TRAFFICABILITY TEST DATA					
PROJECT LOCATION			**TYPE OF VEHICLE**		
CONE INDEX VALUES					
	DIAL READINGS AT DEPTH				
TEST NUMBERS	**0"**	**6"**	**12"**	**18"**	**24"**
1					
2					
3					
4					
5					
AVERAGE					
CI_{50} =					
CI_1 =					
CRITICAL LAYER FOR 50 VEHICLES			**CRITICAL LAYER FOR 1 VEHICLE**		

5. Determine if the soil strength profile is normal or abnormal.

6. Determine CI_1 and CI_{50} (Figure 6-2)

Figure 6-2.

Normal Abnormal CI$_1$ =				
CI$_{50}$ =				
CRITICAL LAYER FOR 1 VEHICLE		CRITICAL LAYER FOR 50 VEHICLES		

7. Use soil from the critical layer to perform a remolding test. (Figure 6-3)

FIGURE 6-3.

REMOLDING INDEX VALUES								
	LAYER		LAYER		LAYER		LAYER	
DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"								
1"								
2"								
3"								
4"								
AVERAGE								
RI= $\frac{A}{B}$								
REMOLDING INDEX FOR 1 VEHICLE				REMOLDING INDEX FOR 50 VEHICLES				

8. Determine the rating cone index(RCI) for 1 and 50 vehicles.(SEE Figure 6-4)

FIGURE 6-4.

RATING CONE INDEX

$RCI_1 = CI_1 \times RI_1 =$

$RCI_{50} = CI_{50} \times RI_{50} =$

9. Find VCI_{50} in Appendix C.

10. Compare RCI_{50} to VCI_{50} to deternine if 50 vehicles can pass. If RCI_{50} is larger than VCI_{50}, 50 vehicles can pass; if RCI_{50} is smaller the 50 vehicles cannot pass.

11. Find VCI_1 in Appendix C.

12. Compare RCI_1 to VCI_1 to determine if 1 vehicle can pass. If RCI_1 is larger then a vehicle can pass; If RCI_1 is smaller then the vehicle cannot pass.

STUDENT NOTES AND CALCULATIONS

TRAFFICABILITY TEST DATA

PROJECT LOCATION Ft. Belvoir, Va.	TYPE OF VEHICLE M113A1

CONE INDEX VALUES

	DIAL READINGS AT DEPTH				
TEST NUMBERS	0"	6"	12"	18"	24"
1	48	56	64	70	
2	42	50	58	64	
3	41	49	57	63	
4	46	54	62	68	
5	44	52	60	66	
AVERAGE					
CI_{50} =					
CI_1 =					

CRITICAL LAYER FOR 50 VEHICLES	CRITICAL LAYER FOR 1 VEHICLE

0"-6"			REMOLDING INDEX VALUES 6"-12"					
DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"	57	50	54	47	55	49		
1"	59	53	56	50	58	52		
2"	62	55	59	51	60	53		
3"	66	57	64	53	65	55		
4"	71	59	67	54	70	56		
AVERAGE								
RI = $\frac{A}{B}$								

REMOLDING INDEX FOR 50 VEHICLES	REMOLDING INDEX FOR 1 VEHICLE

RATING CONE INDEX

$RCI_{50} = RI_{50} \times CI_{50}$

$RCI_1 = RI_1 \times CI_1 =$

VEHICLE CONE INDEX

VCI_{50} = Will 50 Vehicles pass ________

VCI_1 = Will 1 Vehicle pass ________

TYPE OF SOIL: FG XXX CG ____

SOIL STRENGTH PROFILE: NORMAL ____ ABNORMAL ____

TRAFFICABILITY TEST DATA

PROJECT LOCATION Ft. Belvoir, Va.	TYPE OF VEHICLE 2 1/2 Ton M217

CONE INDEX VALUES

	DIAL READINGS AT DEPTH				
TEST NUMBERS	0"	6"	12"	18"	24"
1	58	63	69	73	79
2	63	69	73	75	80
3	65	71	75	77	80
4	72	80	82	87	90
5	75	76	75	79	82
AVERAGE					

CI_{50} =

CI_1 =

CRITICAL LAYER FOR 50 VEHICLES	CRITICAL LAYER FOR 1 VEHICLE

	0"-6"		REMOLDING INDEX VALUES 6"-12"					
DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"	58	71	63	67	71	83		
1"	60	73	63	68	75	85		
2"	63	75	65	71	79	90		
3"	65	77	66	73	80	94		
4"	70	81	67	78	85	97		
AVERAGE								
RI = $\frac{A}{B}$								

REMOLDING INDEX FOR 50 VEHICLES	REMOLDING INDEX FOR 1 VEHICLE

RATING CONE INDEX

$RCI_{50} = RI_{50} \times CI_{50}$

$RCI_1 = RI_1 \times CI_1 =$

VEHICLE CONE INDEX

VCI_{50} = Will 50 Vehicles pass ____________

VCI_1 = Will 1 Vehicle pass ____________

TYPE OF SOIL: FG XX CG ____

SOIL STRENGTH PROFILE NORMAL __ __ ABNOLMAL ____

TRAFFICABILITY TEST DATA

PROJECT LOCATION Ft. Belvoir, Va.	TYPE OF VEHICLE Tractor, wheeled, Clark 290M

CONE INDEX VALUES

	DIAL READINGS AT DEPTH				
TEST NUMBERS	0"	6"	12"	18"	24"
1	51	61	70	78	82
2	57	58	64	72	84
3	54	60	67	73	74
4	53	60	68	75	84
5	61	65	72	70	80
AVERAGE					
CI_{50} =					
CI_1 =					

CRITICAL LAYER FOR 50 VEHICLES	CRITICAL LAYER FOR 1 VEHICLE

	3"-9"		REMOLDING INDEX VALUES 9"-15"				15"-21"	
DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"	61	58	67	65	83	72	148	102
1"	63	59	68	65	88	76	148	108
2"	59	52	71	68	93	79	146	114
3"	56	50	73	69	95	81	148	116
4"	60	52	78	70	97	83	150	104
AVERAGE								
RI = $\frac{A}{B}$								

REMOLDING INDEX FOR 50 VEHICLES	REMOLDING INDEX FOR 1 VEHICLE

RATING CONE INDEX

$RCI_{50} = RI_{50} \times CI_{50}$

$RCI_1 = RI_1 \times CI_1 =$

VEHICLE CONE INDEX

VCI_{50} =

VCI_1 =

Will 50 Vehicles pass ________________

Will 1 Vehicle pass ________________

TYPE OF SOIL: FG XXXX CG ____

SOIL STRENGTH PROFILE: NORMAL ____ ABNORMAL ____

STUDENT NOTES

SOILS AND VERTICAL STRUCTURES

I. Types of soil movement that affect vertical structures.

- A. Soil Swell -
- B. Frost Heave -
- C. Permafrost -
- D. Settlement -
- E. Soil Flow -
- F. Other Conditions -

II. Soil Swell

- A. Factors
  - 1. Moisture
  - 2. Expansive soil
    - a. usually is highly plastic clay
    - b. often is improperly compacted
    - c. proximity to the water table
    - d. poorly drained
    - e. can have a high % of organic material
    - f. can be very porous due to having many voids

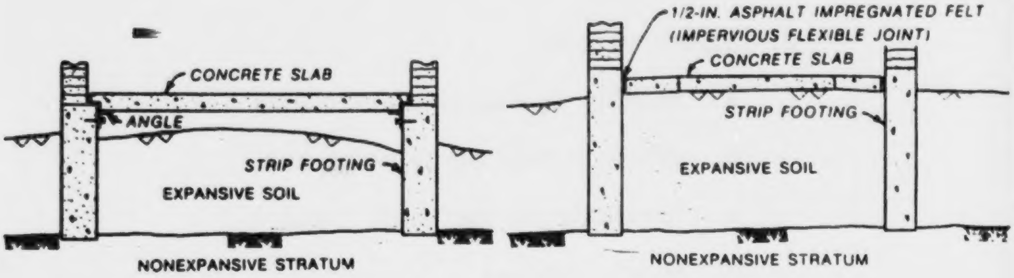
TABLE 7-1. CLASSIFICATION OF SOILS BY PLASTICITY INDICES

SOIL CLASSIFICATION	PLASTICITY INDEX (PI)
non-swelling	0 to 14
marginal swell	14 to 25
critical swell	25 to 40
excessive swell	40 and up

B. Preventive and Corrective Measures

1. Compact properly
2. Provide adequate drainage
3. Replace poor soil
4. Remove vegetation and organic material
5. Stabilize the soil
6. Minimize the number of fills
7. Construct to suit conditions
    a. suspend walls, floors, and service lines
    b. build a ringwall or other extensive foundation
    c. use a post-tensioned cable system
    d. build at another site to avoid problems

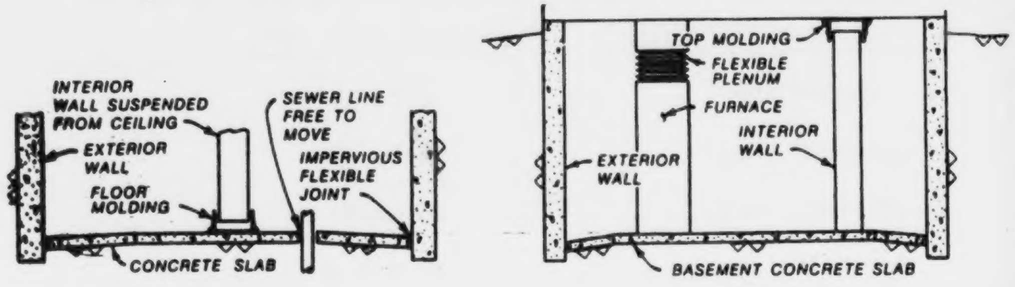
FIGURE 7-1. SUSPENDED FLOORS


CONCRETE SLAB
ANGLE
STRIP FOOTING
EXPANSIVE SOIL
NONEXPANSIVE STRATUM
1/2-IN. ASPHALT IMPREGNATED FELT
(IMPERVIOUS FLEXIBLE JOINT)
CONCRETE SLAB
STRIP FOOTING
EXPANSIVE SOIL
NONEXPANSIVE STRATUM

a. Suspended Structural Floor (See Figures C-8 and C-9 for Details)

b. Floating Floor Slab

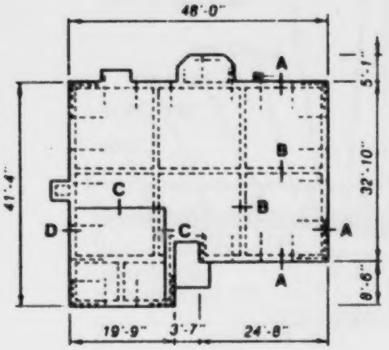
FIGURE 7-2. SUSPENDED WALLS AND SERVICES


INTERIOR WALL SUSPENDED FROM CEILING
EXTERIOR WALL
FLOOR MOLDING
CONCRETE SLAB
SEWER LINE FREE TO MOVE
IMPERVIOUS FLEXIBLE JOINT
TOP MOLDING
FLEXIBLE PLENUM
FURNACE
INTERIOR WALL
EXTERIOR WALL
BASEMENT CONCRETE SLAB

a. Wall Suspended from Ceiling

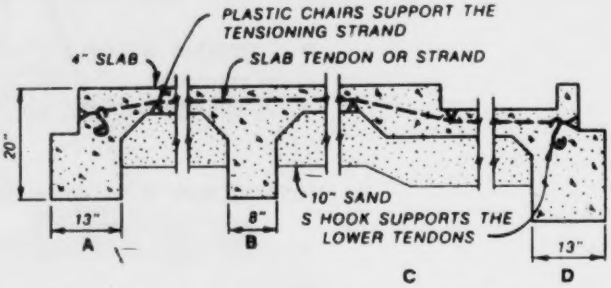
b. Furnace and Interior Wall Supported on Floor

FIGURE 7-3. POST-TENSIONED CABLE SYSTEM


48'-0"
A
5'-1"
B
32'-10"
41'-4"
C
B
D
C
A
A
8'-6"
19'-9"
3'-7"
24'-8"

FOUNDATION:
Type Post-tensioned
P I 30 to 35
Concrete: 3000 psi
Slab Tendons: 1/2" Ø
(6' on center each way approximate)

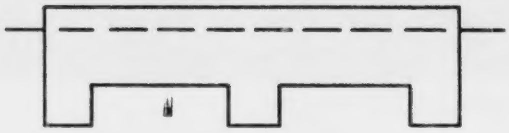
Beam Tendons: 1/2" Ø
(one in perimeter beams)
Fill: 10" sand
Membrane: None


PLASTIC CHAIRS SUPPORT THE TENSIONING STRAND
4" SLAB
SLAB TENDON OR STRAND
20"
10" SAND
S HOOK SUPPORTS THE LOWER TENDONS
13"
A
8"
B
13"
C
D

(Department of Housing and Urban Development, Region IV)

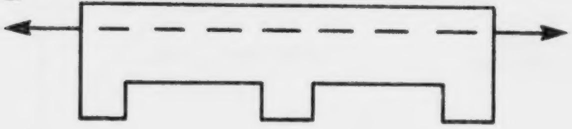
FIGURE 7-4. POST-TENSIONED CABLE SYSTEM

1.



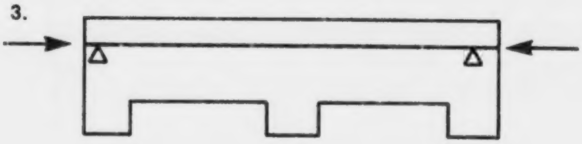
Concrete Placed and Gains Sufficient Strength

2.



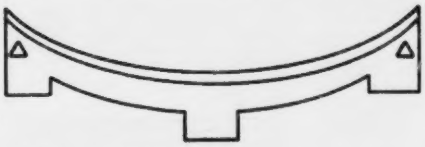
Cables Inserted and Stretched

3.



Cables Mechanically Anchored.

4.



Moment Produced and Mat Desires This State of Equilibrium

III. Cold Weather Construction Problems

A. Frost Heave

1. Required conditions

a. frost susceptible soil -

b. ground water present

c. temperature below freezing

2. Preventive and corrective measures

a. put footings below the frost line

b. remove and replace the soil

c. drain the area

d. add anti-freeze agents to the soil

B. Permafrost

1. Geographic locations -

2. Effects on construction

3. Remedial measures

a. replace with non-frost susceptible soil

b. suspend the floor

c. insulate the floor and ventilate the foundation

d. build on pilings

e. refrigerate the subgrade

FIGURE 7-5. INSULATED FLOOR WITH VENTILATED FOUNDATION

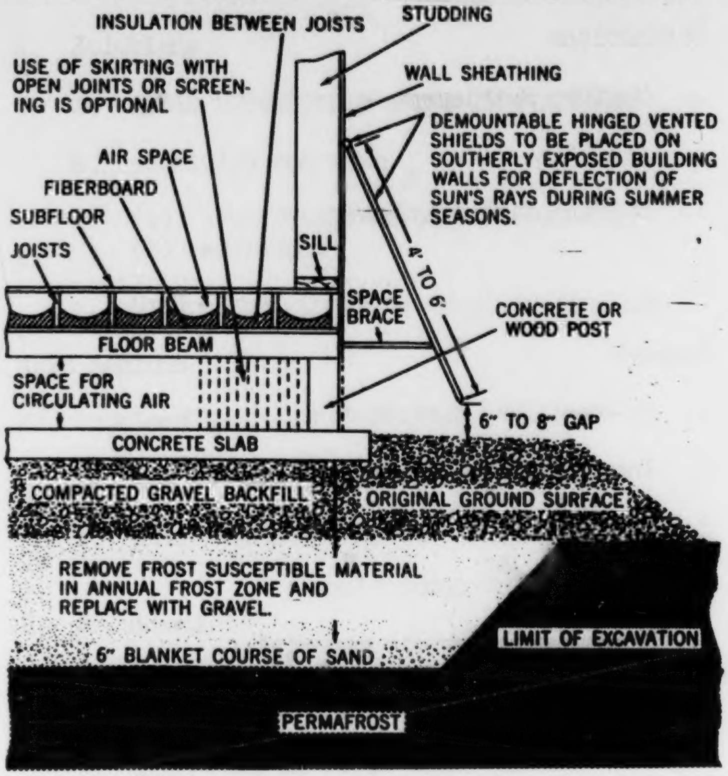
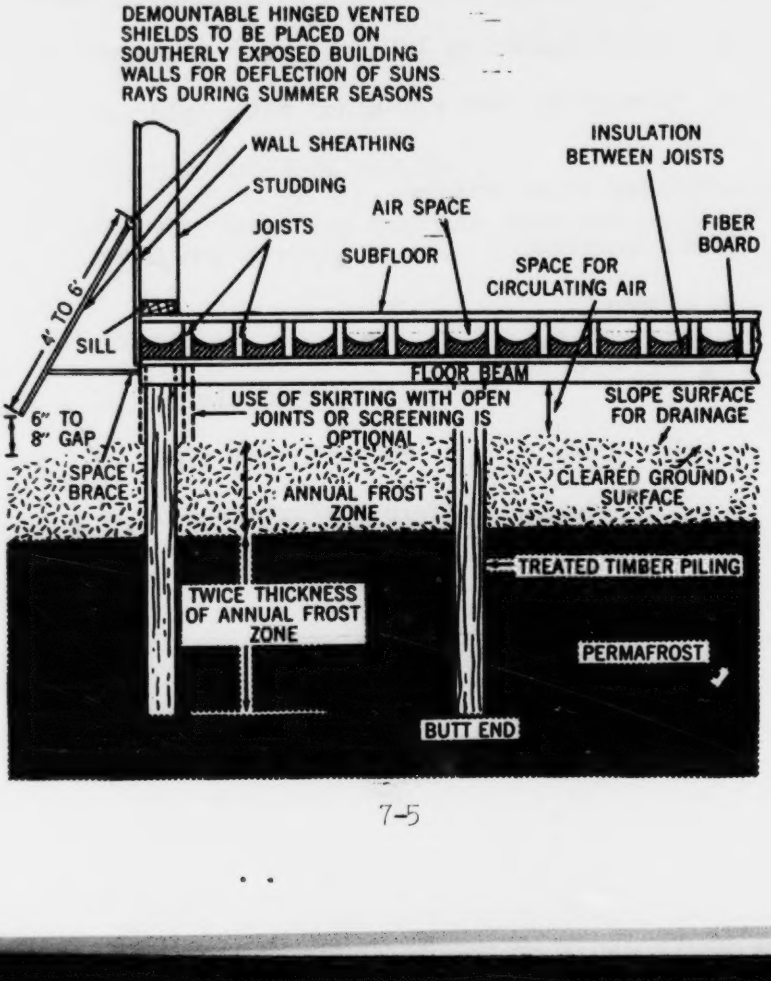

INSULATION BETWEEN JOISTS
STUDDING
USE OF SKIRTING WITH OPEN JOINTS OR SCREENING IS OPTIONAL
WALL SHEATHING
DEMOUNTABLE HINGED VENTED SHIELDS TO BE PLACED ON SOUTHERLY EXPOSED BUILDING WALLS FOR DEFLECTION OF SUN'S RAYS DURING SUMMER SEASONS.
AIR SPACE
FIBERBOARD
SUBFLOOR
JOISTS
SILL
4' TO 6'
SPACE BRACE
CONCRETE OR WOOD POST
FLOOR BEAM
SPACE FOR CIRCULATING AIR
6" TO 8" GAP
CONCRETE SLAB
COMPACTED GRAVEL BACKFILL
ORIGINAL GROUND SURFACE
REMOVE FROST SUSCEPTIBLE MATERIAL IN ANNUAL FROST ZONE AND REPLACE WITH GRAVEL.
LIMIT OF EXCAVATION
6" BLANKET COURSE OF SAND
PERMAFROST

FIGURE 7-6. INSULATED FLOOR ON PILINGS


DEMOUNTABLE HINGED VENTED SHIELDS TO BE PLACED ON SOUTHERLY EXPOSED BUILDING WALLS FOR DEFLECTION OF SUNS RAYS DURING SUMMER SEASONS
WALL SHEATHING
INSULATION BETWEEN JOISTS
STUDDING
AIR SPACE
JOISTS
FIBER BOARD
SUBFLOOR
SPACE FOR CIRCULATING AIR
4' TO 6'
SILL
FLOOR BEAM
6" TO 8" GAP
USE OF SKIRTING WITH OPEN JOINTS OR SCREENING IS OPTIONAL
SLOPE SURFACE FOR DRAINAGE
SPACE BRACE
ANNUAL FROST ZONE
CLEARED GROUND SURFACE
TREATED TIMBER PILING
TWICE THICKNESS OF ANNUAL FROST ZONE
PERMAFROST
BUTT END

IV. Soil Settlement

A. Definitions

1. Uniform Settlement -

2. Differential Settlement -

B. Factors

1. Un-even load distribution
2. Incorrect soil moisture content
3. Incorrect soil type
4. Insufficient compaction
5. Decomposing organic material

C. Prevention

1. Compact properly
2. Avoid unnecessary fills
3. Avoid organic soils
4. Distribute load evenly

V. Soil Flow and Other Problems

A. Earth Movements

1. Types

a. soil creep -

b. slump -

c. landslide -

2. Solutions

   a. AVOID SUSCEPTIBLE AREAS IF POSSIBLE

   b. stabilize the site

      (1) lime/cement/biuminous materials
      (2) membranes

   c. insure that there is adequate drainage

B. Arid Regions

1. Problems

   a. too little moisture in the soil

   b. soil migration

2. Solutions

   a. add water to the soil to achieve OMC, then compact while still wet.

   b. hold soil in place with cements

C. Coastal Regions

1. Problems

   a. high water table

   b. high level of erosion

   c. salt water and salt air cause corrosion of metal and concrete.

2. Solutions

   a. proper drainage

   b. soil stabilization

   c. use corrosion resistant concrete

STUDENT NOTES

APPENDIX A

ANSWERS TO PRACTICAL EXERCISES

FIELD IDENTIFICATION (PAGE 2-14)

1. Fine-grained means that at least 50% of the soil is fines, so you do the field identification on the material that passes the #40 sieve. The fact that this soil can not be rolled into a 1/8" thread and can only be made into a 2" ribbon indicates that this soil is a silt, so it is classified M.

2. The 40% of this soil that does not pass the #4 sieve is gravel. This leaves 60% to be sand and fines. To find the % sand you must multiply the % that settled in the sedimentation test by the total % of sand and fines, then divide by 100 (60%x60%/100=36%). This gives you the result of 36% sand. Sixty % sand and fines minus 36% sand leaves 24% fines. Since 40% gravel is the largest fraction, G is the primary classification letter. Since the field identification tests on the sub-#40 material give you conflicting results (the shine and taste tests indicate clay, but the wet shaking test indicates silt), you choose the soil type indicated by the majority of the tests, in this case clay(C). Thus your final classification is GC.

3. You have 78% sand and fines, as the 22% that does not pass the #4 sieve is gravel(100%-22%=78%). The % sand is found by multiplying the % sand and fines by the % that settles in a sedimentation test, then dividing by 100 (78%x70%/100=54.6%). The 54.6% sand is larger than the 22% gravel so the primary letter is S. The % fines = 100% - 54.6% sand - 22% gravel = 23.4%. When you ran your field ID tests on the sub-#40 material you found that all three tests indicated that the fines were silt, so your secondary letter is M. The final classification is SM.

4. The 60% of this soil that passed the #4 sieve is sand and fines. 100% - 60% = 40% gravel that was retained on the #4 sieve. The 80% of the sand and fines that settled is sand, and since 80%x 60%/100 = 48%, you have 48% sand. 60% sand and fines minus 48% sand leaves 12% fines. The sand fraction(48%)is larger than the gravel fraction(40%), so S is the primary letter. Once again, your results from the field ID tests on the sub-#40 material are conflicting(feel test and hand wash test indicate silt, but the thread test indicates clay), so you choose the majority result which is silt(M). Thus, your final classification is SM.

5. 100% - 45% gravel(given) = 55% sand and fines. The 60% of the sand and fines that settled is sand, so you find that 33% of this soil is sand(55%x60%/100 = 33%). The 45% gravel is greater than the 33% sand so the primary letter is G. 100% soil minus 45% gravel and minus 33% sand leaves 22% fines(100-45-33=22). All three of your field ID tests indicate that the fines are silt, so your secondary letter is M. This gives you a final classification of GM.

6. Since the given amount of fines is greater than 50% this soil is classified as fine-grained. You use the field ID tests to determine which type of fines are present and you find that the dry strength and ribbon tests both indicate high-compressibility clay, so your classification for this soil is CH.

7. The 5% of this soil retained on the #4 sieve is gravel. The remaining 95% is sand and fines. Since 99% of the sand and fines settles in the sedimentation test, the soil is 94% sand, (95%x99%/100% = 94.05% sand) and only 1% is fines. Since the C_u is greater than 6 and the C_c is between 1 and 3 the soil is well graded. The field classification is S but since you know the C_u and C_c you know that the USCS symbol is SW.

8. (Page 2-15)

a. Either soil #2 or soil #6 would be acceptable. Both are quite impermeable, but soil #2 would probably be better because the larger gravel fraction would give it a higher shear strength.

b. Soil #7 is the only acceptable answer, as it is the only one of the soils which has very little to no frost action (SEE Table 1, column 10).

c. You can fill sandbags with almost any material, so you usually choose whatever is easiest to obtain. However, you want the densest and coarsest material you can find so that the bags will be stable and provide maximum protection. Fines can cause problems if they get wet, as they may leak through the pores of the sandbags. Soils # 2,3,4,5, and7 would all be acceptable, but soil #7 would be the best answer.

d. The sand of soil #7 would be the best answer because of its drainage characteristics(SEE Table 1, column 12).

SAND CONE PROBLEMS (PAGE 3-7)

2. 12.63 lbs sand plus mold
-9.5 lbs mold
3.13 lbs sand in mold

3.13 lbs sand in mold / (1/30 ft^3) =93.9 lbs/ft^3(pcf)

3.
a. 4.83lbs sand / 93.9pcf = .0514 ft^3

b. 7.62 lbs / .0514 ft3 = 148.25pcf

c. 148.25pcf / 1.136 = 130.50pcf

PAGE 3-11 (Figure 3-9)

1. OMC = 9%

2. Maximum CE55 γ_d = 125pcf

3. Moisture range is 7% to 11%

4. Density limits after compaction are 118.75pcf to 125pcf.

SOILS STABILIZATION P.E. (PAGE 5-9)

1. a. Soil is SC. ($_d$ =130)

b. Cement is used, at an 8% concentration(S%).

c. Area = 50' x 4800' = 240,000ft^2

Lift thickness(T) = 6", with 5 lifts for a 30" total

Q(for 1 lift) = A x T/12 x γ_d x S% = 240,000 x 6/12 x 130 x 8/100

=1,248,000lbs per lift.

Total Q (for all 5 lifts) = 5 x 1,248,000 = 6,240,000lbs.

Number of bags/lift = 1,248,000lbs/94lbs/bag =13277 bags.

total number of bags = 5 x 13277 = 66385 bags.

The area per bag =240,000ft^2/13277 bags = 18.1ft^2/bag, therefore the bags are placed $\sqrt{18.1ft^2}$, or 4.25'apart.

SOIL STABILIZATION (Continued) Page 5-10

2. a. Lime is needed for this GC soil, in a 4% concentration.

b. γ_d = 140pcf   Area = 10' x 9000' = 90,000ft^2

Q(for 1 lift) = A x T/12 x γ_d x S% =

90,000ft^2 x 6"/12" x 140pcf x 4%/100% =

252,000lbs/lift = 2520 bags/lift

Using a 10% waste factor, the final number of bags/lift is 2520 x 1.10 = 2772 bags/lift.

The total number of bags required = 2772 bags/lift x 7 lifts = 17640 bags total.

The area per bag = 90000ft^2/ 2772 bags = 32.5ft^2/bag

The spacing of the bags is $\sqrt{32.5}$ = 5.7' center to center.

3. a. The correct stabilizer is cement at 8% concentration.

b. γ_d = 130pcf   Area = 70' x 5200' = 364,000ft^2

Q(for 1 lift) = A x T/12 x γ_d x S%/100 =

364,000 x 6/12 x 130 x 8/100 =

1,892,000lbs/lift.

1,892,000 lbs/lift x 7 lifts = 13,244,000lbs total

1,892,000 lbs/ lift / 94lbs/bag = 20137 bags/lift

20137 bags/lift x 7 lifts = 140,959 bags total.

Area per bag = 364,000ft^2/20137 bags = 18.1 ft^2/bag, so the bags are spaced $\sqrt{18.1}$ = 4.25' apart.

STUDENT NOTES

PAGE 6-9

TRAFFICABILITY TEST DATA

PROJECT LOCATION Ft. BELVOIR, Va.	TYPE OF VEHICLE M113A1

CONE INDEX VALUES

	DIAL READINGS AT DEPTH				
TEST NUMBERS	0"	6"	12"	18"	24"
1	48	56	64	70	
2	42	50	58	64	
3	41	49	57	63	
4	46	54	62	68	
5	43	51	59	65	
AVERAGE	44	52	60	66	
CI_{50} = 56	48	56	63		
CI_1 = 48		52	59.5 or 60		

CRITICAL LAYER FOR 50 VEHICLES 6"-12"	CRITICAL LAYER FOR 1 VEHICLE 0"-6"

REMOLDING INDEX VALUES

	0"- 6"		6"-12"					
DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"	57	50	54	47	55	49		
1"	59	53	56	50	58	52		
2"	62	55	59	51	60	53		
3"	66	57	64	53	65	55		
4"	71	59	67	54	70	56		
AVERAGE	63.00	54.80	60.00	51.00	61.60	53.00		
RI = $\frac{A}{B}$	$\frac{54.80}{63.00}$ =.87		$\frac{51.00}{60.00}$ =.85		$\frac{53.00}{61.60}$ =.86		$\frac{.85+.86}{2}$ =.86	

REMOLDING INDEX FOR 50 VEHICLES .86	REMOLDING INDEX FOR 1 VEHICLE .86

RATING CONE INDEX

$RCI_{50} = RI_{50} \times CI_{50}$ = .86 x 56 = 48.16

$RCI_1 = RI_1 \times CI_1$ = .87 x 48 = 41.76

VEHICLE CONE INDEX

VCI_{50} = 40

VCI_1 = 17

Will 50 Vehicles pass YES

Will 1 Vehicle pass YES

TYPE OF SOIL: FG XXX CG _____

SOIL STRENGTH PROFILE NORMAL XXX ABNORMAL _____

PAGE 6-10

TRAFFICABILITY TEST DATA

PROJECT LOCATION Ft. Belvoir, Va.	TYPE OF VEHICLE 2 1/2 Ton M217

CONE INDEX VALUES

	DIAL READINGS AT DEPTH				
TEST NUMBERS	0"	6"	12"	18"	24"
1	58	63	69	73	79
2	63	69	73	75	80
3	65	71	75	77	80
4	72	80	82	87	90
5	75	76	78	81	82
AVERAGE	67	72	75	79	82

CI_{50} = 74 CI_1 = 70	69.5 ♂70	73.5 ♂74	77	

CRITICAL LAYER FOR 50 VEHICLES 6"-12"	CRITICAL LAYER FOR 1 VEHICLE 0"-6"

	0"-6"		REMOLDING INDEX VALUES		6"-12"			
DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"	58	71	63	67	71	83		
1"	60	73	63	68	75	85		
2"	63	75	65	71	79	90		
3"	65	77	66	73	80	94		
4"	70	81	67	78	85	97		
AVERAGE	63.20	75.40	64.80	71.40	78.00	89.80		
RI = $\frac{A}{B}$	$\frac{75.40}{63.20}$=1.19		$\frac{71.40}{64.80}$=1.09		$\frac{89.80}{78.00}$=1.15		$\frac{1.09+1.15}{2}$ =1.12	

REMOLDING INDEX FOR 50 VEHICLES 1.12 = 1	REMOLDING INDEX FOR 1 VEHICLE 1.19 = 1

RATING CONE INDEX

$RCI_{50} = RI_{50} \times CI_{50} = 1 \times 74 = 74$

$RCI_1 = RI_1 \times CI_1 = 1 \times 70 = 70$

VEHICLE CONE INDEX

VCI_{50} = 62

VCI_1 = 27

Will 50 Vehicles pass YES

Will 1 Vehicle pass YES

TYPE OF SOIL: FG XX CG ____

SOIL STRENGTH PROFILE: NORMAL XX ABNORMAL ____

TRAFFICABILITY TEST DATA

PROJECT LOCATION Ft. Belvoir, Va.	TYPE OF VEHICLE Tractor, wheeled, Clark 290M

CONE INDEX VALUES

	DIAL READINGS AT DEPTH				
TEST NUMBERS	0"	6"	12"	18"	24"
1	51	61	70	78	82
2	57	58	64	72	84
3	54	60	67	73	74
4	53	60	68	75	84
5	61	65	72	70	80
AVERAGE	55.20↘55	60.80↗61	68.2↘68	73.6↗74	80.8↗81
CI_{50} = Normal 68, Abnormal 75	58	64.5↗65	71	77.5↗78	
CI_1 = 62, 62		61.5↗62	68	74.5↗75	

CRITICAL LAYER FOR 50 VEHICLES 9"-15" CRITICAL LAYER FOR 1 VEHICLE 3"-9"

REMOLDING INDEX VALUES

	3"-9"		9"-15"				15"-21"	
DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"	61	58	67	65	83	72	148	102
1"	63	59	68	65	88	76	148	108
2"	59	52	71	68	93	79	146	114
3"	56	50	73	69	95	81	148	116
4"	60	52	78	70	97	83	150	104
AVERAGE	59.80	54.20	71.40	67.40	91.20	78.20	148.00	108.80
RI = $\frac{A}{B}$	$\frac{54.20}{59.80} = .91$		$\frac{67.40}{71.40} = .94$		$\frac{78.20}{91.20} = .86$		$\frac{108.80}{148.00} = .74$	

REMOLDING INDEX FOR 50 VEHICLES NORMAL $\frac{.94+.86}{2} = .90$ ABNORMAL .74	REMOLDING INDEX FOR 1 VEHICLE .91

RATING CONE INDEX

$RCI_{50} = RI_{50} \times CI_{50}$ = NORMAL .90x68 = 61.20 ABNORMAL .74x75 = 55.50

$RCI_1 = RI_1 \times CI_1$ = NORMAL/ABNORMAL = .91x62 = 56.42

VEHICLE CONE INDEX

VCI_{50} = 77

VCI_1 = 34

Will 50 Vehicles pass NO

Will 1 Vehicle pass YES

TYPE OF SOIL: FG XXXX CG ____

SOIL STRENGTH PROFILE NORMAL ____ ABNORMAL XXXX

STUDENT NOTES

APPENDIX B

(TM 5-330
CHAPTER 9)

SOILS TRAFFICABILITY

Section I. INTRODUCTION

9-1. Introduction

This chapter contains information on soils trafficability, the capacity of soils to support specific military vehicles. It describes the standard issue cone penetrometer and the soil sampling and remolding equipment and explains their care and use. A section is devoted to making trafficability estimates from terrain data, mainly soils data. The information in this chapter for making trafficability estimates is a part of a more comprehensive Army mobility prediction system that determines the combined effects of all terrain attributes, including soils, on vehicle performance when operating cross-country. See reference below.

9-2. Principles Considered

The chapter emphasizes the trafficability of fine-grained and coarse-grained soils and does not consider the two other predominant earth surface materials, organic soils (muskeg) and snow. Procedures are presented for: (a) measuring trafficability by tests on the soil in place with a cone penetrometer, soil sampler, and remolding equipment; and (b) estimating trafficability from a knowledge of terrain data (topography, soil type, etc.) and weather conditions. The chapter is limited to problems offered by the soils themselves; it does not include problems offered by natural or man-made obstacles such as forests or ditches, nor

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Reference - Technical Report No. 11789 (LL143) "The AMC '71 Mobility Model" TACOM-WES, July 1973.

does it include treatment of vehicle characteristics such as the maximum tilt or side-angle at which a vehicle can climb without power stall or overturning. As mentioned in paragraph 9-1, the combined effects of terrain features such as soil, vegetation, and slope can be estimated through use of the more complex Army mobility prediction system. The fundamental principles underlying the procedures presented are sound for temperate and tropical climates and for soils that have been subjected to freeze-thaw cycles, if they are not frozen at the time of testing and passage of traffic.

9-3. Terminology

Certain terms peculiar to soils trafficability are defined as follows:

a. Bearing Capacity. The ability of a soil to support a vehicle without undue sinkage of the vehicle.

b. Traction Capacity. The ability of a soil to resist the vehicle-tread thrust required for steering and propulsion.

c. Cone Index. An index of the shearing resistance of soil obtained with the cone penetrometer; a number representing resistance to penetration into the soil of a 30-degree cone with 1/2-square-inch base area (actually load in pounds on cone base area in square inches).

d. Remolding. The changing or working of a soil by traffic, or by a remolding test. Remolding may have a beneficial, neutral, or detrimental effect, resulting in a change of soil strength.

e. Remolding Index. The ratio of remolded soil strength to original strength, determined in accordance with procedures to be described in paragraph 9-11. Soil conditions that permit the remolding test to be run usually are such that a loss of strength rather than an increase of strength results.

f. Rating Cone Index. The measured cone index multiplied by the remolding index; it expresses the soil-strength rating of a point subjected to sustained traffic.

g. Critical Layer. The soil layer in which the rating cone index (fine-grained soil) or cone index (coarse-grained soil) is considered the most significant measure of trafficability. Its depth varies with weight and type of vehicle, as well as with the soil profile. Generally, the critical layer for different soil types is defined as follows:

(1) Fine-grained soils

(a) 6 to 12 inches below the surface when subjected to 50 passes of a vehicle.

(b) Surface to 6-inch depth when subjected to one pass of a vehicle.

(2) Coarse-grained soils: surface to 6-inch depth for all vehicular passes.

h. Vehicle Cone Index (VCI). The index assigned to a given vehicle that indicates the minimum soil strength in terms of rating cone index (or cone index for coarse-grained soil) required for one pass (VCI_1) or 50 passes (VCI_{50}) of the vehicle.

i. Stickiness. The ability of a soil to adhere to vehicles.

j. Slipperiness. Low traction capacity of a soil's surface owing to its lubrication by water or mud without the occurrence of significant vehicle sinkage.

k. Mobility Index. A number that results from a consideration of certain vehicle characteristics.

l. Maximum Towing Force. The maximum continuous towing force or pull a vehicle can exert (expressed as a ratio or percentage of its own weight).

m. Fine-Grained Soil. A soil of which more than 50 percent by weight of the grains will pass a No. 200 sieve (smaller than 0.074 mm in diameter).

n. Coarse-Grained Soil. A soil of which more than 50 percent by weight of the grains will be retained on a No. 200 sieve (0.074 mm and larger in diameter). For trafficability purposes, these are beach and desert soils usually containing less than 7 percent of material passing the No. 200 sieve, or soils containing 7 percent or more of material passing the No. 200 sieve but not in a wet condition (remoldable; see below).

o. Remoldable Sands. A coarse-grained soil, usually containing 7 percer or more of material passing a No. 200 sieve, in which the water content greatly influences the trafficability characteristics and on which the remolding test can be performed. When wet, these soils react to traffic in a manner similar to fine-grained soils except that they are more sensitive to remolding.

9-4. Basic Trafficability Concepts

a. Soil Strength. Both the bearing and traction capacities of soils are functions of their shearing resistance. Shearing resistance is measured by the cone penetrometer and is expressed in terms of cone index. Since the strength of a fine-grained soil may increase or decrease when loaded or disturbed, remolding tests (para 9-11) are necessary to measure the gain or loss of soil strength to be expected under traffic. A comparison of the rating cone index (para 9-3f) with the vehicle cone index (para 9-3h) indicates whether the vehicle can negotiate the given soil condition for a given number of passes. For example, if a soil has a cone index of 120 and a remolding index of 0.60 in its critical layer, its strength may be expected to fall to 120 times 0.60, or a rating cone index of 72, under traffic. Accordingly, it is considered that such a soil is not trafficable for vehicles with vehicle cone indexes greater than 72.

b. Stickiness. The operation of vehicles in wet fine-grained soil sometimes can be hampered seriously by stickiness. Under extreme conditions, sticky soil can accumulate in the running gear of a vehicle to the point where travel and steering are difficult. Normally, stickiness is troublesome only when it occurs in soils of low bearing capacity (normally fine-grained soils).

c. Slipperiness. The presence of excess water or a layer of soft plastic soil overlying a firm layer of soil can produce a slippery surface. Such a condition may make the steering of all vehicles difficult or immobilize rubber-tired vehicles. Vegetation, especially when wet and on a slope, may cause immobilization of rubber-tired vehicles. Slipperiness can be troublesome even when associated with soils of high bearing capacity.

d. Variation of Trafficability With Weather. Weather changes will produce changes in the trafficability of a soil. In rainy periods, fine-grained soils undergo an increase in moisture, with resultant increased slipperiness and stickiness and decreased strength. Dry periods have opposite effects. Loose sands improve in trafficability through an increase in cohesion as a result of rainy periods and return to the loose, less trafficable, state during dry periods. It is apparent that trafficability characteristics measured on a given date cannot be applied later unless full allowance is made for the changes in soil strength caused by weather. Techniques have been developed for predicting the effects of weather on soil trafficability. These techniques are part of the comprehensive Army mobility prediction system and are not included in this chapter.

Section II. INSTRUMENTS AND TESTS FOR TRAFFICABILITY

9-5. Test Set, Soil (Trafficability)

Trafficability measurements are made with the aid of the soil test set (trafficability). This set consists of the following items: one canvas carrying case, one cone penetrometer with 5/8-inch aluminum staff and 0.5-square-inch cone, one soil sampler, remolding equipment (includes 3/8-inch steel staff and 0.2-square-inch cone, 5/8-inch steel staff with foot, handle, 2-1/2-pound hammer, cylinder and base with pin), a bag of hand tools (2 open-end wrenches 1/2 x 9/16, one 6-inch Stillson wrench, 1 combination spanner wrench and 1/4-inch screwdriver, one 3/16-inch Allen wrench and one 2-inch screwdriver with 1/8-inch bit). The items are shown in their proper places in the carrying case in figure 9-1. The set is carried on the back, as shown in figure 9-2. The weight of the complete set is 19 pounds.

Figure 9-1. Test set, soil trafficability

Figure 9-2. Carrying test set, soil trafficability

9-6. The Cone Penetrometer

The cone penetrometer (figs. 9-3 and 9-4) is the principal instrument used in evaluating soils trafficability. It consists of a 30-degree cone of 1/2-square-inch base area, an aluminum staff 19 inches long and 5/8 inch in diameter, a proving ring, a micrometer dial, and a handle. When the cone is forced into the ground, the proving ring is deformed in proportion to the force applied. The amount of force required to move the cone slowly through a given plane is indicated on the dial inside the ring. This force is considered to be an index of the shearing resistance of the soil and is called the cone index of the soil in that plane. The range

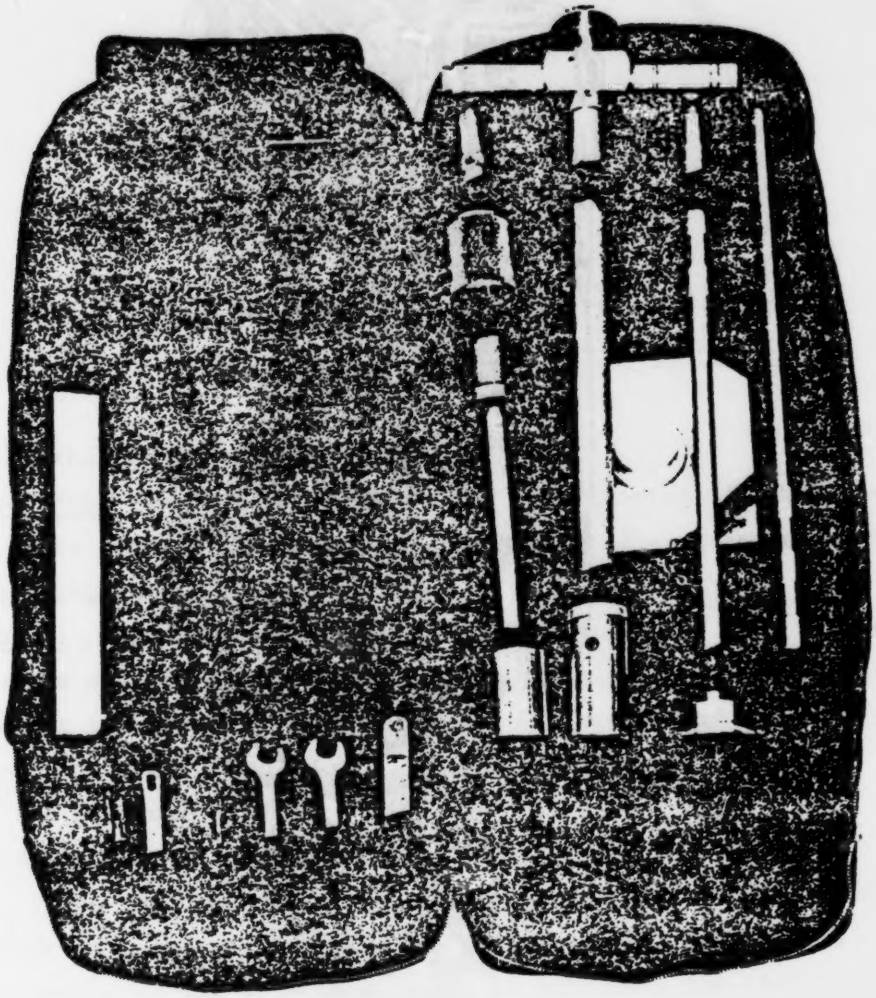


Figure 9-1. Test set, soil trafficability.

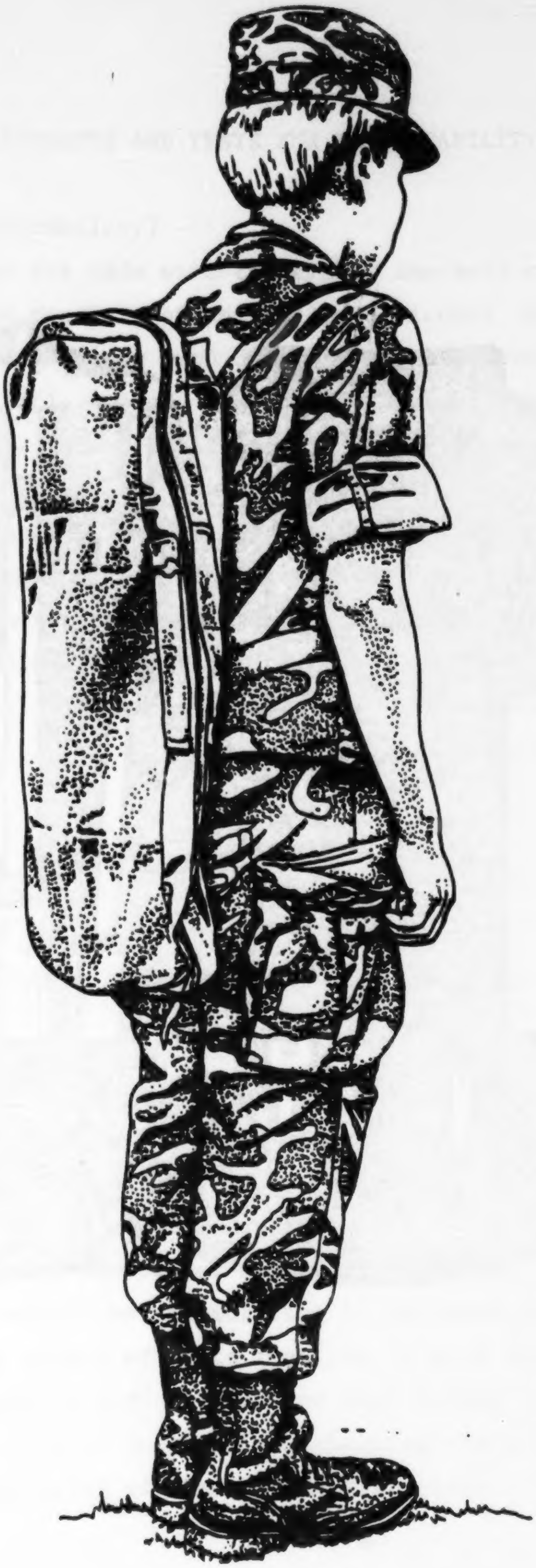


FIGURE 9-2. Carrying test set, soil trafficability

of the dial is 0 to 300 (150 pounds). The proving ring and handle are used with a 3/8-inch-diameter, 19-inch-long steel staff for remolding tests in remoldable sands (para 9-11c).

Figure 9-3. Cone penetrometer

Figure 9-4. Cone penetrometer in use
Upright position

Figure 9-4. Cone penetrometer in use
Prone position

9-7. Use of the Penetrometer

a. Inspection. Inspect the penetrometer before using to make sure that all nuts, bolts, and joints are tight and that the dial-gage stem contacts the proving-ring bearing block.

b. Zeroing. Allow the penetrometer to hang vertically from its handle and rotate the dial face until "0" is under the needle. Note that when the instrument is kept vertical between the fingertips and allowed to rest on its cone, the dial will register about 4, or 2 pounds, the total weight of the instrument.

c. Operation.

(1) Place the hands over each other on the handle, palms down, and approximately at right angles, as shown in figure 9-4, to minimize eccentric loading of the proving ring and to help keep the staff vertical.

(2) Apply force until slow, steady downward movement occurs.

(3) Take a dial reading just as the base of the cone is flush with the ground surface. To do this, watch cone descend until an instant before the cone base is expected to be flush with the ground surface, then immediately shift the vision to the dial face. Continue the slow, steady, downward movement and take successive dial readings at appropriate intervals, usually at 6-inch intervals (see para 9-18 for guidance on

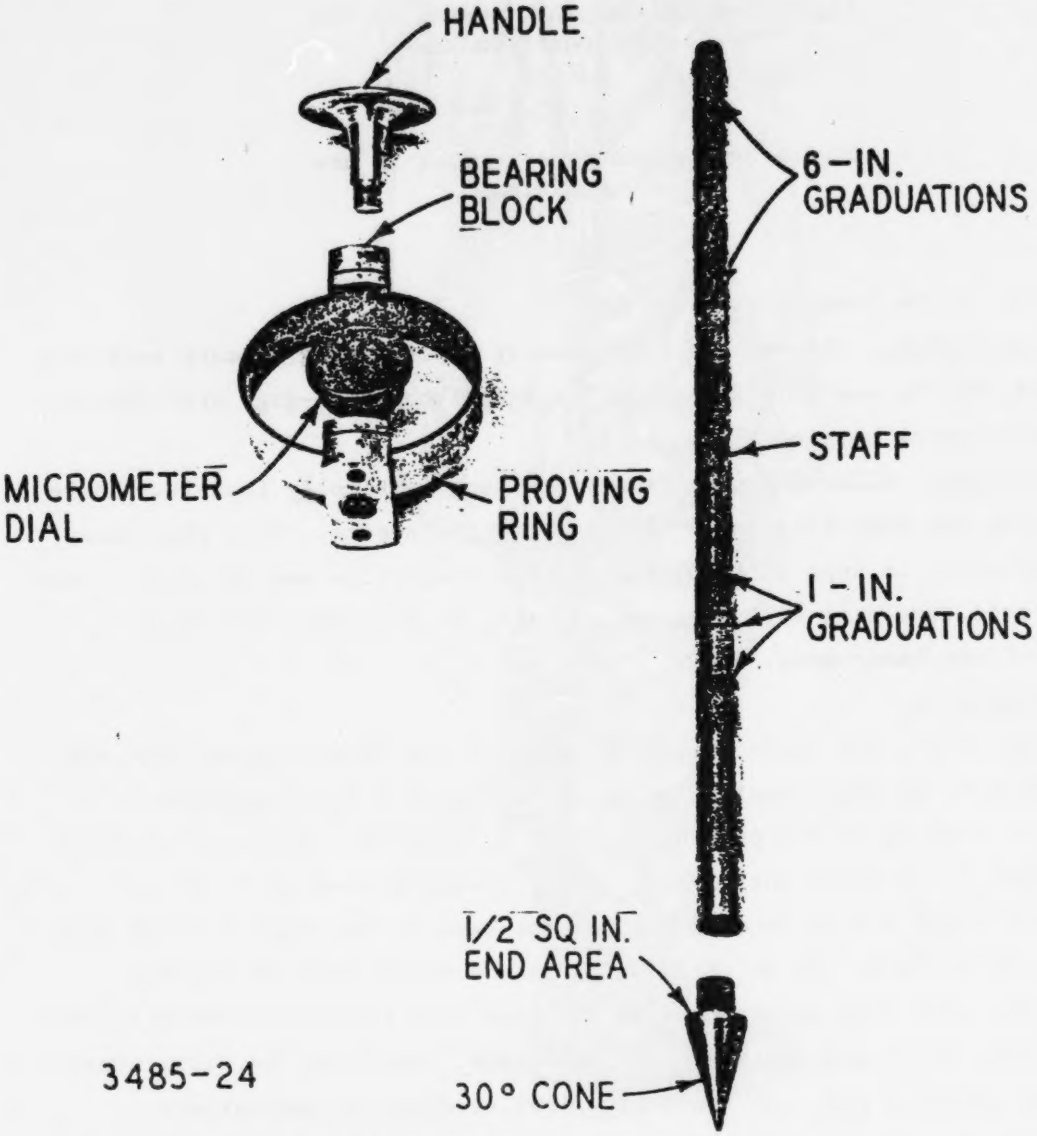

HANDLE
BEARING BLOCK
6-IN. GRADUATIONS
MICROMETER DIAL
PROVING RING
STAFF
1-IN. GRADUATIONS
1/2 SQ IN. END AREA
3485-24
30° CONE

Figure 9-3. Cone penetrometer.

FIGURE 9-4a. Prone position

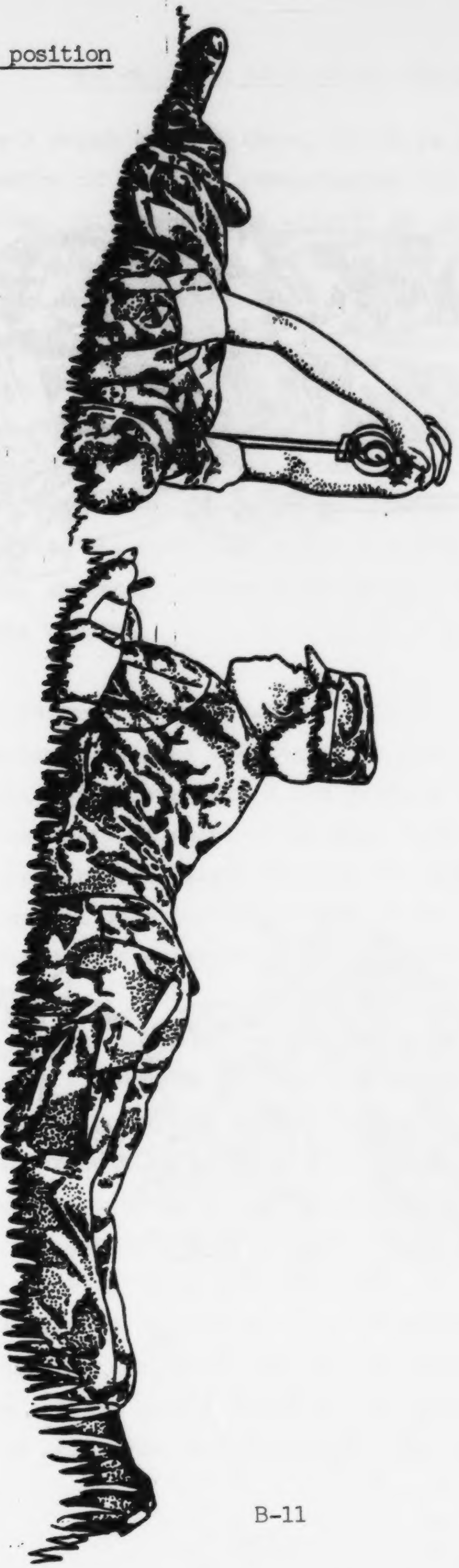
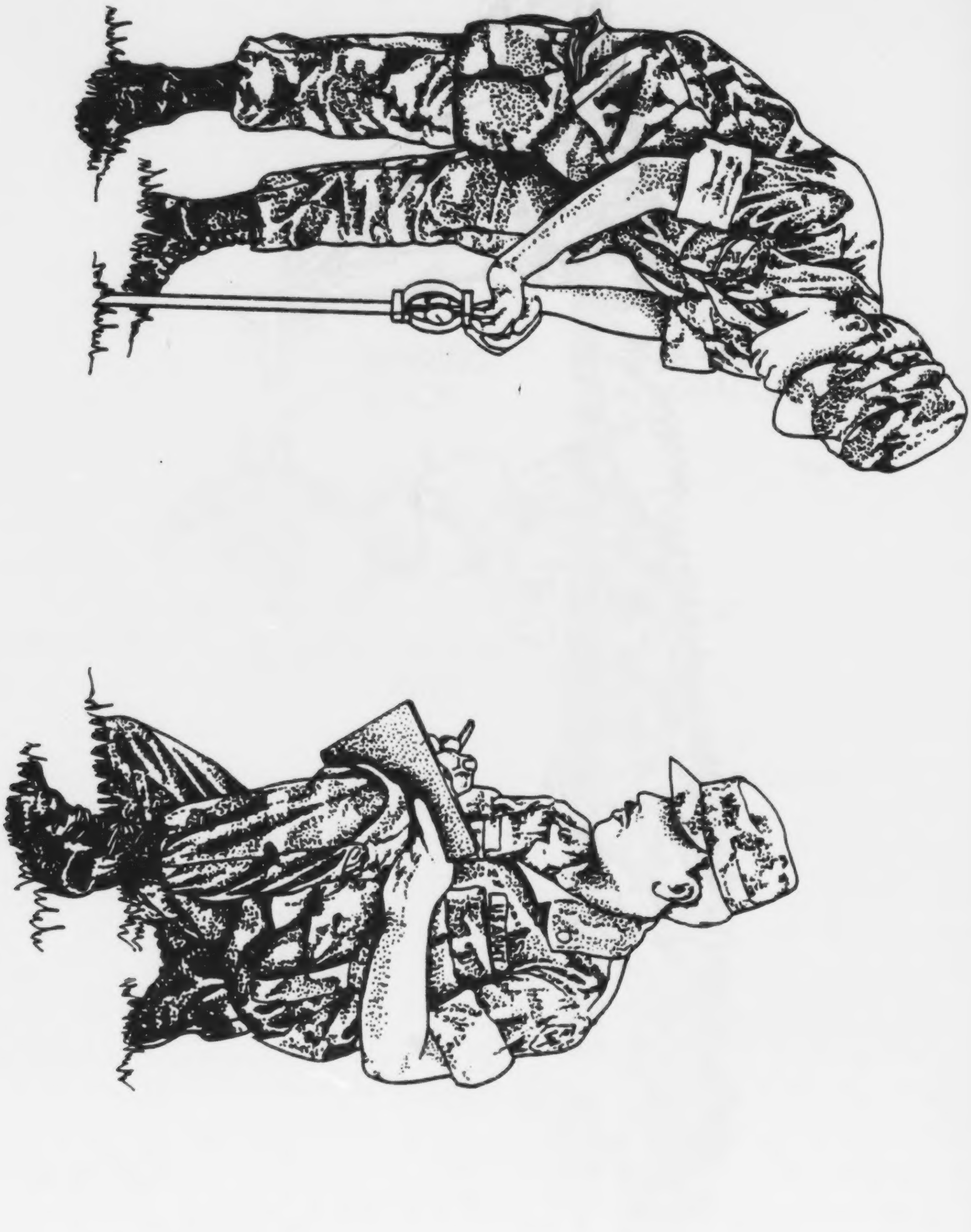


FIGURE 9-4b. Upright position for cone penetrometer



proper intervals) to a depth of 18 inches. If it is necessary to stop the downward progression of the cone penetrometer for any reason, the progress may be resumed with no adverse effects on cone penetrometer readings. For example, when only one man is on the trafficability reconnaissance, he may find it convenient to make two cone penetrometer readings, stop the penetration to record the readings, resume the penetration to obtain two additional readings, stop and record, and so on.

Note. The use of an assistant to the operator increases the speed with which measurements can be made and recorded and usually diminishes the likelihood of errors. Some two-man teams prefer that the assistant merely record the values the operator verbally transmits, while other teams prefer that the assistant announce the proper depth to the operator as well as record the reading.

d. Cautions.

(1) Keep the instrument vertical.

(2) Do not attempt to make readings that are higher then the capacity of the dial since this might overstress the proving ring.

(3) If dial capacity is exceeded at less than 18 inches of penetration, make another penetration nearby because the cone might be striking an isolated rock fragment or other hard, small object.

(4) Do not withdraw the instrument by the ring, but always by the staff or the handle.

(5) Be very careful to read the cone index at the proper depth. If readings are actually made as little as one-quarter inch from the proper depth and recorded as being at the proper depth, an average of such readings will not accurately reflect the average strength at that depth. Carelessness in making proper depth determination is probably the greatest source of error in the use of the penetrometer.

9-8. Training Penetrometer Operators

Operators should be trained in an area of uniform soil conditions. The instructor should take approximately 50 sets of readings spaced equally over the area. The average cone indexes for 6-inch layers should be computed and used as standards or references. The trainee should be instructed in all proper techniques of operation and should practice penetration under the eye of a qualified instructor until he has become familiar with the techniques of operation. He then should make about 50 sets of readings, using an assistant to record them. The average cone indexes obtained by a trainee should then be compared to the standard. If the trainee's readings deviate widely, the causes for the deviation should be sought and corrected. In a uniform area, 5% deviation is considered wide. The most probable cause of error is carelessness in determining the proper depth.

The rate of progression recommended is such that four readings (surface, 6-, 12-, and 18-inch) can be measured in 15 seconds in a continuous penetration in a soft soil. Much slower or faster rates of penetration will reflect lower or higher values, respectively, but the discrepancies will not be large. Effects on cone index of variation in rate of penetration for the same operator or even between experienced operators are insignificant. The possibility of mechanical imperfections of the cone penetrometer should be investigated if deviations are persistent. Sometimes a needle sticks on a loose dial face or slips on its shaft. Sometimes dial faces are jarred or otherwise rotated around the shaft of the dial, causing an improper zero setting. A damaged or overstressed ring might even require recalibration. A micrometer dial stem may not have been in good contact with the proving-ring bearing block when the instrument was zeroed.

9-9. Care and Adjustment of the Penetrometer

a. General Care. Little care is required beyond keeping the penetrometer free from dirt and rust, keeping all parts tight, and frequently checking and, if necessary, rezeroing the instrument. Particular care should also be taken to see that no grit is caught between the stem of the dial and the lower mounting block.

b. Dial. The micrometer dial is a sensitive instrument that should be protected against water and rough usage. It should never be immersed in water and should be wiped dry as soon as possible after its use in rainy weather. When transported by truck, the dial should be cushioned by wrapping it in paper or cloth.

c. Bearing-Block Adjustments. If either or both bearing blocks should become loosened and moved, they should be adjusted so that they lie on the same diameter of the ring and retightened, and the proving ring recalibrated. Calibration while on reconnaissance is not feasible. All readings made in the field after bearing blocks have had to be removed should be noted and corrected according to the calibration made later.

d. Cone Replacement. Considerable use of the same cone may result in a rounding of its point. This will not affect the accuracy of the instrument, but if the base of the cone has had excessive wear or is deformed by hard usage, the cone should be replaced.

e. Proving-Ring Recalibration. The calibration will remain true for the life of the instrument unless the bearing blocks are actually moved or the ring is severely overstressed, deformed by a hard knock, or subjected to extreme changes in temperature or other unusual strains. If the ring needs recalibration, the following steps should be taken:

(1) Remove handle and staff.

(2) Place lower mounting block of the ring assembly on a smooth, horizontal surface.

(3) Check bearing-block alinement and tightness. Both blocks should be on the same diameter of the ring. A drafting triangle or a carpenter's square may be used in this operation. The bolts should be snug.

(4) The stem of the dial should bear firmly on the lower block with sufficient travel of the arm available for the full range (approximately 1/10-inch deflection) of the proving ring. The dial can be moved up or down by adjusting the two nuts on the threaded stud that holds the gage in position. Both nuts should be tight when in final position.

(5) Zero the dial by rotating its face so that "0" is under the needle (para 9-7b).

(6) Add load in 10-pound increments up to 150 pounds, marking or noting the position of the needle on the dial after the addition of each load increment. Any of the following loading methods may be used:

(a) Deadweights may be added to the top of the ring assembly. If a plate is used to hold the weights, its weight should be considered in the first 10-pound load.

(b) Any of the load machines commonly used in laboratory work may be used to apply the load.

(c) The ring assembly may be placed on a set of platform scales and the load increments applied by a jack and measured with the platform scales.

(7) Remove load in 10-pound increments, noting the position of the needle after the removal of each increment.

(8) The load run should be made at least twice, using the average of the needle position for each increment as the final point.

(9) Some variation in needle position will occur, but will not be significant.

(10) When 10-pound intervals have been established on the face of the dial, they may be marked 20, 40, etc., to 300. Each interval may then be subdivided into four subintervals. Each interval should be subdivided separately since the arcs for various 10-pound intervals are not necessarily the same.

9-10. Soil Sampler

A piston-type soil sampler (fig. 9-5) is used to extract soil samples for remolding tests (para 9-11).

a. Use. Hold the disk at the top of the piston rod firmly with one hand to prevent vertical movement of the piston, and force the sampling tube into the soil with the other hand (top photo, fig. 9-6). In firm soils, two men often are needed to force the sampler into the soil. After locking the piston rod by turning the knurled handle, twist the instrument slightly and withdraw. Deposit the sample directly into the remolding cylinder (para 9-11b). The bottom photo on figure 9-6 shows the technique for using the sampler in a prone position. While it is preferable to hold the disk stationary with one hand, in a prone position two hands are usually needed to force the sampler into the soil.

b. Care. It is essential to keep the inside of the sampling tube, the piston ring, and the leather washer reasonably clean. After five to twenty-five samplings, depending upon the type of soil, immerse the tube first in water and then in fuel oil, working the piston up and down five or six times in each liquid. Wipe off the excess fuel oil, then squirt light machine oil into the tube. If the instrument becomes stiff and hard to work, remove the tube, disassemble, and thoroughly clean the piston and oil the leather washer. Tube walls and cutting edges are comparatively soft and should be handled with some care.

c. Adjustment. The effective piston-rod length should be adjusted to keep the face of the piston flush with the cutting edge of the tube when the piston rod handle (disk) is fully depressed. This is done by loosening the setscrew on the handle, screwing the handle up or down to the correct position, and retightening the setscrew.

Figure 9-5. Soil Sampler

Figure 9-6. Soil sampler in use upright position

Figure 9-6. Soil sampler in use prone position

9-11. Remolding Test

a. Equipment. The equipment for the remolding test, shown in detail in figure 9-7 and in use in figure 9-8, consists of a steel cylinder approximately 2 inches in diameter and 8 inches long, mounted on an aluminum base, a 2-1/2-pound steel drophammer sliding on an 18-inch steel staff with handle, and a cone penetrometer. The cone penetrometer may be equipped with either the aluminum shaft with the 0.5-square-inch cone (for fine-grained soils) or more slender steel shaft with the 0.2-square-inch cone (for remoldable sands). The penetrometer is used to measure

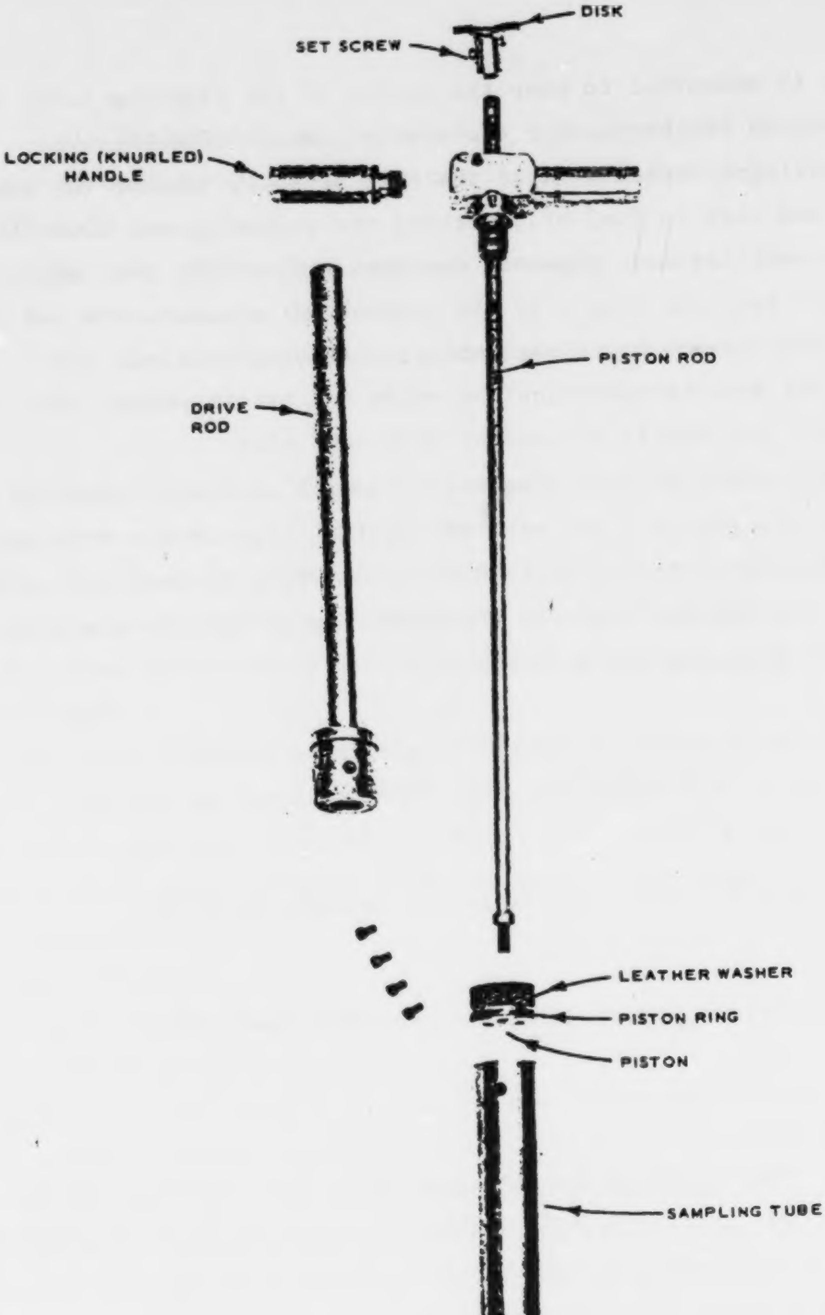

DISK
SET SCREW
LOCKING (KNURLED) HANDLE
PISTON ROD
DRIVE ROD
LEATHER WASHER
PISTON RING
PISTON
SAMPLING TUBE

Figure 9-5. Soil sampler.

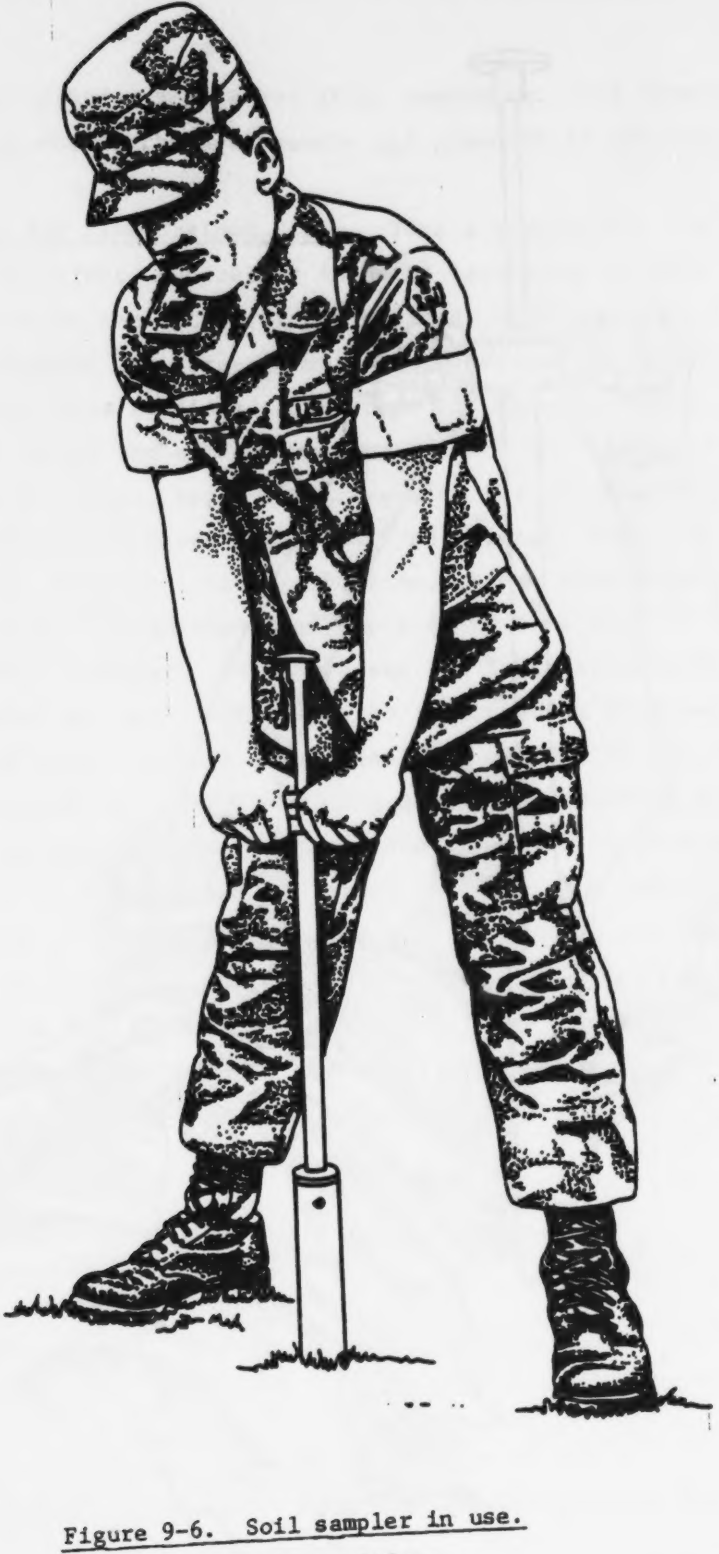
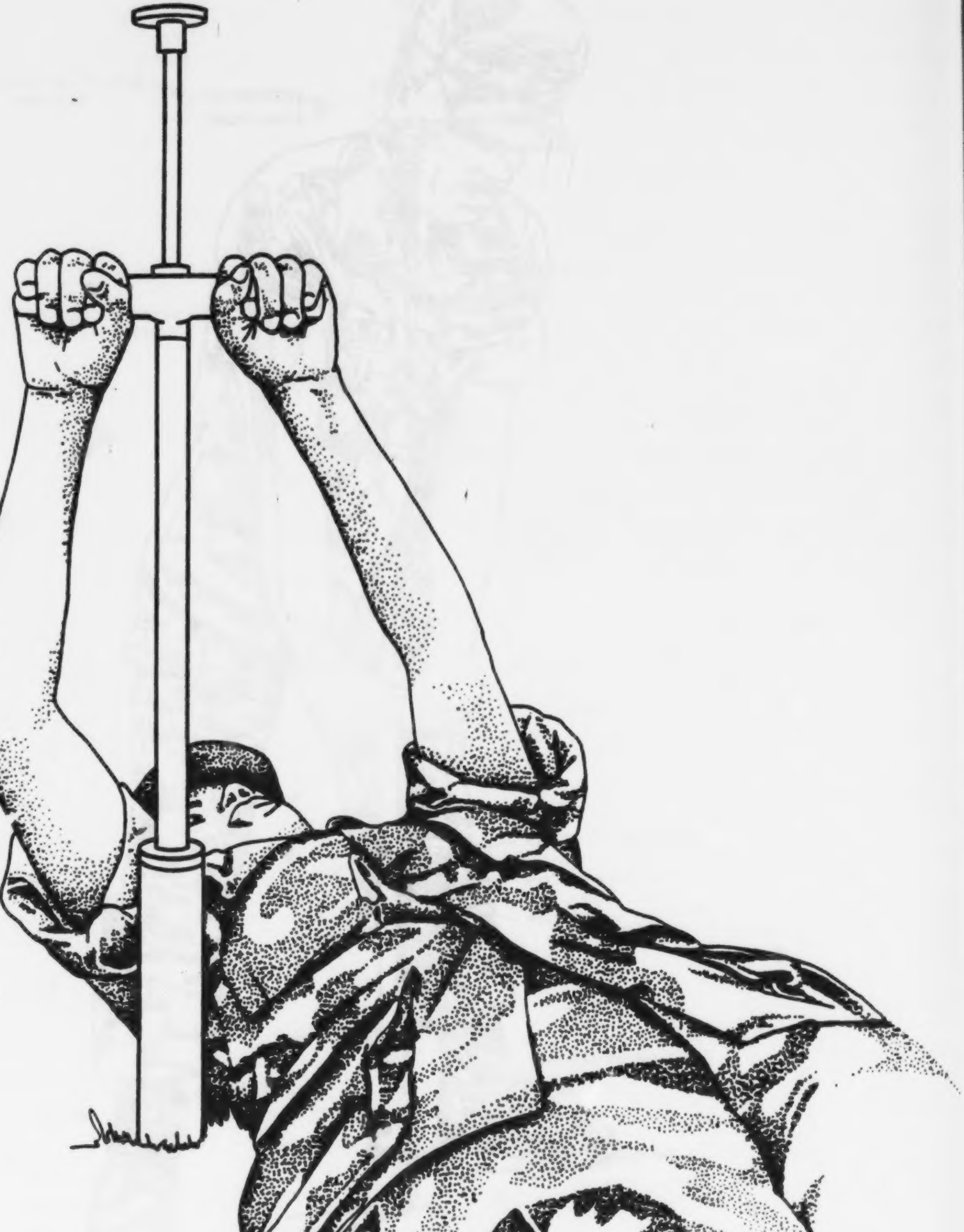


Figure 9-6. Soil sampler in use.

Upright position

FIGURE 9-6b. Prone position



soil strength in the cylinder before and after remolding. The sampler (fig. 9-5) is used to obtain the soil sample and place it in the remolding cylinder.

b. Test Procedure for Fine-Grained Soils. Take a sample with the sampler (1, fig. 9-8), eject it directly into the remolding cylinder (2, fig. 9-8), and push it to the bottom of the cylinder with the foot of the drophammer staff. Measure the strength with the penetrometer (aluminum staff) by taking cone index readings as the base of the cone enters the surface of the soil sample and at each successive inch to a depth of 4 inches (3, fig. 9-8). Next, apply 100 blows with the drophammer falling 12 inches (4, fig. 9-8), and measure the remolded strength from the surface to the 4-inch depth at 1-inch increments, as was done before remolding (3, fig. 9-8). Occasionally, a sample is so hard that it cannot be penetrated the full 4 inches. In such cases the full capacity of the dial (300) is recorded for each inch below the last reading obtained. The sum of the five cone index readings after remolding divided by the sum of the five cone index readings before remolding gives the remolding index.

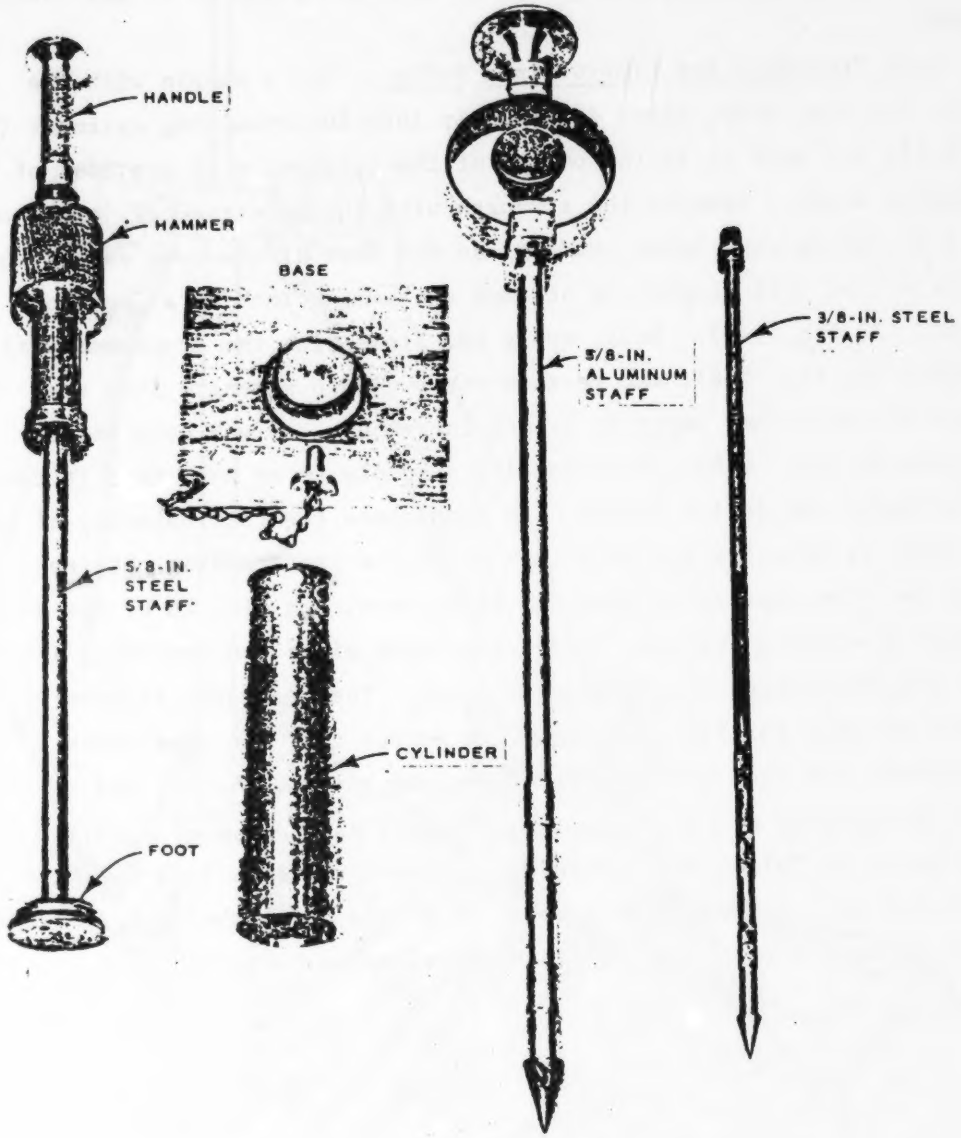
c. Test Procedures for Remoldable Sands. The procedure is generally the same as that for fine-grained soils except that the cone index measurements are made with the slender staff and small cone, and the sample is remolded by placing a rubber stopper in the top of the tube and dropping it (along with cylinder and base) 25 times from a height of 6 inches onto a firm surface, such as a piece of timber. Procedures for identifying remoldable sands are described in paragraph 9-16.

Figure 9-7. Remolding test equipment

Figure 9-8. Remolding test operation

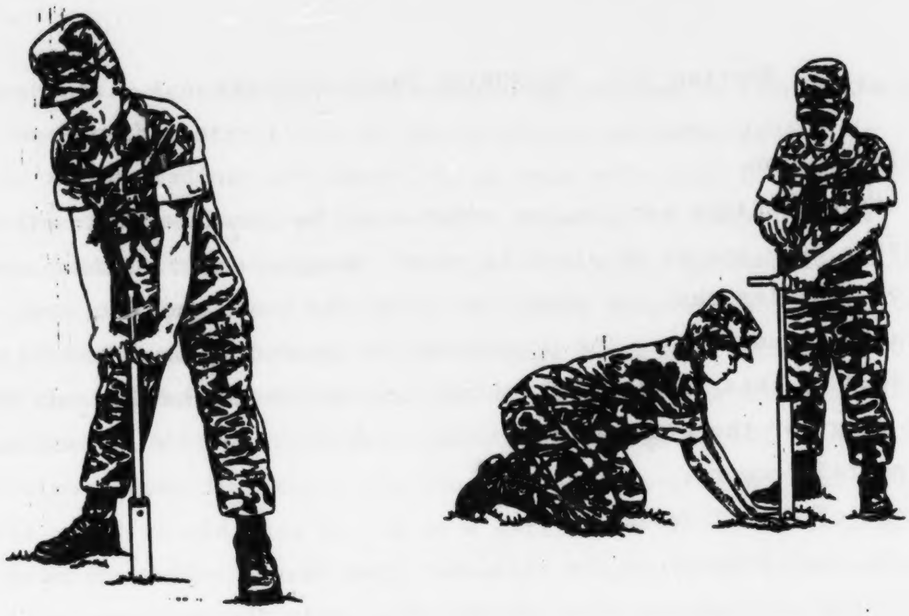
9-12. Other Soil Tests

Shaking tests, plasticity tests, and other field identification tests are described fully in section III, chapter 2.

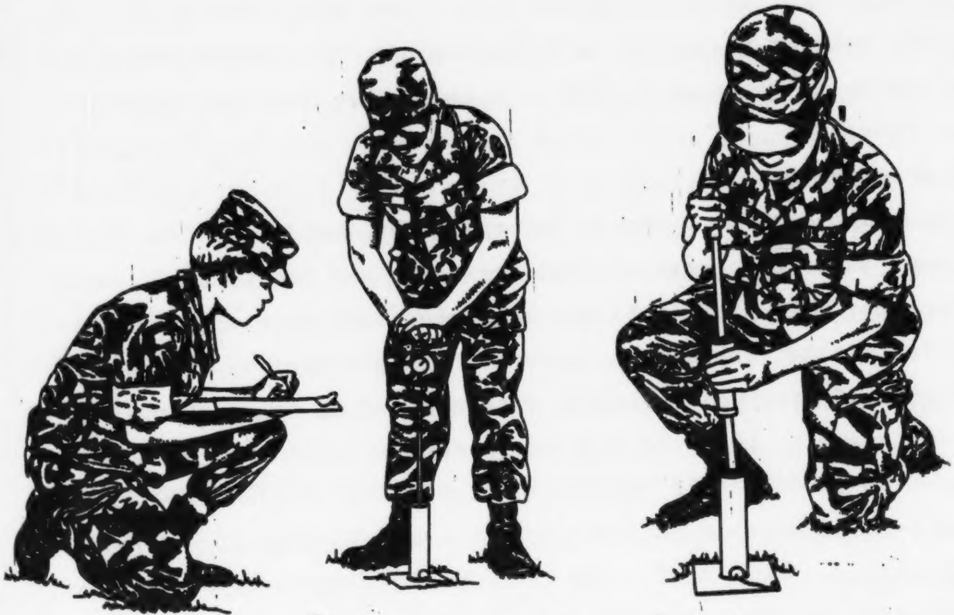

HANDLE
HAMMER
BASE
5/8-IN. STEEL STAFF
FOOT
CYLINDER
5/8-IN. ALUMINUM STAFF
3/8-IN. STEEL STAFF

3485-23

Figure 9-7. Remolding test equipment.



1. Taking sample 2. Loading remolding cylinder



3. Measuring cone index in remolding cylinder 4. Applying hammer blows

Figure 9-8. Remolding test operation.

Section III. MEASURING TRAFFICABILITY

9-13. Introduction

Whenever reconnaissance parties can enter areas to obtain trafficability measurements, data can be obtained to permit determination of the number and type of vehicles that can cross the area, the loads they can tow, and the slopes they can climb. The procedures for measuring trafficability are described in this section. It should be remembered that measurements are valid only for the time of measurement and short periods thereafter, provided no rain occurs.

9-14. Range of Cone Indexes

The range in cone indexes in the critical layer (para 9-18) that is of maximum interest is between about 10 and 300. Only the most mobile of standard military vehicles and some special-purpose experimental vehicles can travel one pass on soils whose cone index is as low as 10, and only a few special vehicles require cone indexes over 300 for 50 passes. An 180-pound man can walk with relative ease on soil whose cone index is 12. These limits usually make it possible, in gathering data for trafficability evaluation, to classify large areas as above or below the critical range without extensive testing.

9-15. Number of Measurements

The number of measurements to be taken is determined by available time tempered by judgment as to the range of soil strengths and the general uniformity of the area. The trafficability measuring instruments have been designed to facilitate rapid observations. The accuracy of the average of any series of readings increases with the number taken. It has been found, for example, that the variations in a typical soft soil are such that about 15 readings are needed to establish a true average cone index at any particular spot at a given depth, and that if 15 readings are made within a radius of 3 feet in a uniform-appearing area, the addition of another reading will not significantly change the average. It is not anticipated that time will be available for this large number of measurements. Accordingly, judgment should be used to reduce the number, in accordance with instructions in the following subparagraphs.

a. Only a few readings are needed in an area with cone indexes above or below the critical range of 10 to 300. However, if cone indexes are within this critical range or, more particularly, between about 10 and 150, many readings should be taken to assure complete and accurate coverage of the area. Readings should be made at enough locations to establish the boundaries of the area and the average cone index within fairly close limits. At least three sets of readings should be made at each location. Remolding tests (in the case of fine-grained soil and remoldable sand) should also be run at a sufficient number of locations to establish the range of remolding indexes. In addition, if a tentative route can be selected in the field, penetrometer and remolding measurements should be made at closely spaced intervals along it to locate any soft spots.

b. Where cone index ranges from 150 to 200, sufficient locations should be selected to verify the limits of the area as established by visual inspection. Three or four sets of readings should be made at each location. For fine-grained soils and remoldable sands, remolding tests should be made at the first two or three locations; if these show a remolding index of 0.90 or more, no additional remolding tests need be made. If the remolding index is below 0.90, and especially if it is much below, a sufficient number of remolding tests should be made to establish the range for the area. Generally, this can be established with tests at about six locations.

c. Where cone indexes are above 200, a very few penetrometer readings normally will suffice to verify the extent of the area. Two sets of profile readings at each of a few locations should be adequate, and remolding tests on soil from the critical layer (fine-grained soils and remoldable sands) should be made at the first two or three locations. If these show a remolding index of 0.80 or more, no additional remolding tests need be made. Sufficient tests should be made to establish the range for the area if the remolding index is below 0.80, and especially if it is much below. Generally, this can be established with tests at about four locations.

d. Where cone indexes are below 10, readings should be limited to the number needed to establish the limits of the nontrafficable area. No remolding tests are required.

9-16. Remolding Test

Since remolding test techniques for fine-grained soils differ somewhat from those for remoldable sands, the operator should be able to recognize the two types of soil for conditions where remolding tests must be made. In such conditions both soil types are wet in appearance and wet to the touch. If squeezed and rolled between the fingertips, the fine-grained plastic soil will feel soft and smooth because such soil particles are small and flat in shape. The other soil type will have a definite abrasive feel because of the presence of the larger, rounder particles of sand. However, there will be many cases in which the operator cannot confidently distinguish the two types. In such cases, both types of remolding tests should be made and the remolding indexes obtained compared. If the lower remolding index is the one obtained with the remolding test for remoldable sands, it may be assumed that the soil is a remoldable sand, and the test for this soil type should be employed throughout the area under investigation. It is emphasized that a good rule to follow in cases of doubt is to run both types of tests and use the lower remolding index.

9-17. Rating Cone Index

The rating cone index (para 9-3f) is the cone index that will result under traffic, and is the final cone index evaluation of a given area. Assuming that the cone index for an area is 85 and the remolding index is 0.80, the rating cone index then would be 85 × 0.80 = 68.

9-18. Critical Layer

As indicated in paragraph 9-3g, the depth of the critical layer varies with soil type, the soil's strength profile, the vehicle type and weight, and number of passes required.

a. Variations with Soil Type, Vehicle Weight and Type, and Number of Passes Required. Variations with soil type, vehicle weight and type, and number of passes required are as follows:

	Depth of Normal Critical Layer, in.			
	One Pass		50 Passes	
Type of Vehicle	F-G Soils*	C-G Soils**	F-G Soils*	C-G Soils**
Sleds	0 to 3	0 to 3	0 to 3	0 to 3
M116 and other tracked vehicles with ground contact pressures less than 4 psi	0 to 6	0 to 6	3 to 9	0 to 6
Wheeled, up to 50,000 lb	0 to 6	0 to 6	6 to 12	0 to 6
Wheeled, over 50,000 lb	3 to 9	0 to 6	9 to 15	0 to 6
Tracked, up to 100,000 lb	0 to 6	0 to 6	6 to 12	0 to 6
Tracked, over 100,000 lb	3 to 9	0 to 6	9 to 15	0 to 6

b. Normal Strength Profile in Fine-Grained Soil and Remoldable Sands. In a soil with a normal strength profile, the cone index readings either increase or remain constant with each increment of depth. An example of an area with a normal strength profile is given in Table 9-1. Cone indexes should be measured at 6-inch increments down to 18 inches in the early stages of reconnaissance in an area. If these measurements consistently reveal that the profile is normal, only readings in the critical layer need be recorded thereafter. For a tracked vehicle weighing less than 100,000 pounds, such as the M48 tank, readings are recorded for the 6- and 12-inch depths. In a normal profile, remolding tests should be run only on samples taken from the normal critical depth for the vehicle in question, since a decrease in remolding index with increasing depth is not common. The rating cone index for this layer is used as the criterion of trafficability for this particular vehicle.

c. Abnormal Strength Profile in Fine-Grained Soil and Remoldable Sand. In an abnormal strength profile, at least one cone index reading is lower than the reading immediately preceding it. An example of an area with an abnormal strength profile is given in Table 9-1. When it has been

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* Fine-grained soils and remoldable sands.

** Coarse-grained soils.

Table 9-1. Examples of Normal and Abnormal Soil Strength Profiles

Fine-grained Soils and Remoldable Sands

Depth in.	Cone Index Penetration Test No.															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	AV

Area A (Normal Strength Profiles)

Surface	28	33	32	27	39	31	30	27	29	30	28	31	33	30	32	30
6	47	48	50	51	49	52	53	49	47	48	50	50	53	51	52	50
12	69	67	71	70	69	67	72	73	68	70	71	68	70	73	72	70
18	80	82	83	83	81	70	80	79	77	78	81	79	77	82	83	80

The results from 3 remolding tests made in the 6- to 12-in. layer are: 0.86, 0.91, and 0.93. The average remolding index for the layer is 0.90.

Area B (Abnormal Strength Profiles)

Surface	32	30	33	31	28	30	29	27	30	31	29	27	32	33	28	30
6	77	78	75	73	76	75	73	78	77	72	74	75	76	72	74	75
12	42	43	45	46	44	47	48	44	42	43	45	46	47	47	46	45
18	38	37	34	36	35	33	32	36	37	37	36	35	33	32	34	35

The results from 3 remolding tests made in the 6- to 12-in. layer are: 0.87, 0.90, and 0.93. The average remolding index for the 6- to 12-in. layer is 0.90. The results from 3 remolding tests made in the 12- to 18-in. layer are: 0.88, 0.91, and 0.91. The average remolding index for the 12- to 18-in. layer is 0.90.

Coarse-grained Soils

Depth in.	Cone Index Penetration Test No.										
	1	2	3	4	5	6	7	8	9	10	AV
Surface	15	20	30	40	15	20	25	30	20	20	24
3	80	60	80	70	50	75	70	60	65	75	68
6	120	110	130	110	120	115	125	115	120	125	120
9	180	175	190	175	180	185	190	180	175	180	181
12	200	220	195	205	210	205	200	210	215	220	208
15	250	230	245	230	250	245	240	235	250	240	242
18	300+	250	280	295	300+	275	280	300+	290	290	284

established that an abnormal strength profile exists, cone index readings should be made and recorded at 6-inch increments from the top of the normal critical layer (6-inch depth for the M48 tank) to 6 inches below the bottom of the normal critical layer (18 inches for the M48 tank). Remolding tests must be run on samples taken from the normal critical layer and also from the 6-inch layer below. The lower rating cone index is used as the measure of trafficability. Sleds are an exception to this rule. The 0 to 3-inch layer is always used as the critical layer for these vehicles.

d. Strength Profile in Coarse-Grained Soil. As indicated in the tabulation in a above, the external layer for most vehicles in coarse-grained soils is the 0- to 6-in layer. The vast majority of coarse-grained soils have a normal strength profile with a rather large increase in strength with depth when compared to fine-grained soils. For this reason cone index measurements should be made at 3-inch increments to 18 inches, or until the maximum capacity (300 cone index) of the penetrometer has been reached. Usually fewer penetrations are required to establish an average since coarse-grained soil areas usually are more uniform than fine-grained soils and remoldable sands. No remolding index tests are required. An example of strength measurements in a coarse-grained soil area is given in Table 9-1.

e. Interpolating Values. Intermediate values for the 3-, 9-, and 15-inch depths (fine-grained soil and remoldable sands) can be interpolated whenever the vehicle types under consideration require them.

f. Examples. It is desired to investigate the following fine-grained soil areas for trafficability for vehicles that are of such type and weight that the 6- to 12-inch depth is the normal critical layer. Since Area A in Table 9-1 has a normal profile, a remolding test was run only for the 6- to 12-inch layer. The rating cone index for Area A is 60 (average of 50 and 70) × 0.90 = 54. In Area B, remolding tests were necessary for both the 6- to 12-inch and 12- to 18-inch layers. In this area, the rating cone index of the 6- to 12-inch layer is 60 (average of 75 and 45) × 0.90 = 54 and of the 12- to 18-inch layer is 40 × 0.90 = 36. The rating cone index of the 12- to 18-inch layer, 36, is the governing value for trafficability in Area B.

9-19. Other Factors

In addition to the cone index of an area, other factors should be considered in evaluating trafficability, as discussed in the following subparagraphs.

a. Slope. The steepest slope, or ruling grade, that must be negotiated should be measured or may be determined from the study of a contour map. For travel over slopes, the cone index requirements must be increased over those required for level terrain. Detailed procedures for determining slope effects are contained in sections IV and VI of this chapter.

b. Stickiness. No instrument for measuring the effects of stickiness on the performance of vehicles has yet been devised. Stickiness will occur in all fine-grained soils when they are comparatively wet. The greater the plasticity of the soil, the more severe are the effects of stickiness. In general, stickiness will have adverse effects on the speed and facility of travel and steering of all vehicles, but will not in itself cause the immobilization of any vehicle except the smallest of tracked vehicles. Even the worst conditions of stickiness are nothing more than a nuisance to the larger, more powerful military vehicles. Removal of fenders will reduce stickiness effects on some vehicles.

c. Slipperiness. Like stickiness, the effects of slipperiness cannot be measured quantitatively. Soils that are covered with water or a layer of soft plastic soil usually are slippery and often cause steering difficulty, especially to rubber-tired vehicles. Immobilization can occur in many instances. Immobilizations occur frequently when slipperiness is associated with low bearing capacity. Slipperiness effects assume greater significance on slopes, and sometimes slopes whose soil strength is adequate may not be passable because of slipperiness. The use of chains on rubber-tired vehicles usually will be of benefit in slippery conditions. The following three categories are used to rate slipperiness:

Condition	Symbol
1. Not slippery under any condition......................	N
2. Slippery when wet	P
3. Slippery at all times	S

d. Vegetation. The particular effects of vegetation on trafficability are not within the scope of this manual, but some points are worthy of mention. Dense grass, especially if wet with dew or rain, may provide slippery conditions. Soil strength requirements will be greater than normal if small trees or thick brush must be pushed down by the vehicle.

e. Organic Soil Areas. Much of the terrain in northern latitudes is blanketed by a layer of organic material composed of roots, mosses, and other vegetation in various stages of decomposition. Limited testing with military vehicles thus far reveals that vehicles such as the M116 and other low ground-pressure tracked vehicles can travel 50 passes over mats that are over 6 inches thick; usually, high ground-pressure vehicles can travel only a few passes before they break through and become immobilized. Wheeled vehicles cannot travel on most of these organic soil areas. Cone indexes give general indications of relative strength of organic soils, but soil strength-vehicle performance relations are not as well defined as for fine-grained and coarse-grained soils.

f. Other Obstacles. A complete assessment of the trafficability of a given area must include an evaluation of obstacles such as forests, rivers, boulder fields, ditches, hedgerows, etc. Exact effects of such obstacles on the performance of vehicles can be determined by the comprehensive Army mobility prediction system, but are not within the scope of this chapter.

Section IV. DETERMINING VEHICLE PERFORMANCE IN FINE-GRAINED SOILS AND REMOLDABLE SANDS

9-20. Background

The procedures and criteria presented in section III are intended for use in tactical operations. The criteria have been established so that when the rating cone index (RCI) of a given area is equal to or higher than the vehicle cone index for 1 and 50 passes (VCI_1 or VCI_{50}) of the using vehicle, sufficient strength will be available in the soil to withstand the passage of 1 or 50 of the same vehicles (or vehicles with smaller VCI_1's or VCI_{50}'s) operating at slow speeds in the same ruts (in the case of 50 passes), and to permit stopping and resumption of movement if necessary. A larger number of vehicles can be moved through the area if the space is available to permit spreading the traffic. If it is desirable or necessary to enter the area, stop, back out of the ruts while turning, and retreat from the area, then the rating cone index of the area should be at least 5 units higher than the vehicle cone index for one pass. Criteria to establish soil strength requirements for 50 passes in the same ruts are slightly different from criteria to establish soil strength requirements for one pass. The main difference is in RCI-vehicle performance (required towing force) relations (Figures 9-9 through 9-12). In the 50-pass criteria, towing force requirements are related only to RCI and the determination of VCI_{50} is not needed; whereas in the 1-pass criteria, required towing force - RCI relations require the determination of VCI_1 as a starting point. The determination of VCI_1 for towed trailers and self-propelled vehicles with non-powered wheels is beyond the scope of this manual. Therefore, examples of 1-pass vehicle performance involving vehicles towing other vehicles always refer to the towed vehicle as an "inoperable powered vehicle." The following paragraphs give examples of the application of vehicle performance criteria for both 50 passes and 1 pass, using Appendix C and Figures 9-9 through 9-12.

9-21. 50-Pass Performance of Self-Propelled Tracked and All-Wheel-Drive Vehicles on Level Terrain

The ability of 50 given vehicles to travel in a straight-line formation on a level area (or the ability of a given vehicle to complete 50 straight-line passes in the same rut) is assured if the rating cone index (RCI) of the area is greater than the vehicle cone index (VCI_{50}). The vehicle cone indexes of many vehicles are listed in Appendix C. When RCI is slightly less than VCI_{50}, there is a possibility that less than 50 vehicles will cross the area before the following vehicles become immobilized. When RCI is considerably less than VCI_{50}, only a few vehicles (if any) will cross the area. Furthermore, free water on the surface of a soil with a cone index of 20 or less in the top few inches (with firm soil immediately below) may cause the soil surface to become slippery which, in turn, may cause the immobilization of wheeled vehicles.

a. Example. Determine whether an M60 tank can complete 50 passes in a level fine-grained soil whose cone index is 70 and remolding index is 0.80 in the critical layer.

From Appendix C: $VCI_{50} = 48$.

$RCI = 70 \times 0.80 = 56$.

Since $RCI > VCI_{50}$, or $56 > 48$, the M60 tank can complete 50 passes.

Section V. CLASSIFICATION OF VEHICLES FOR OPERATION IN FINE-GRAINED SOILS AND REMOLDABLE SANDS

9-30. Classes

Appendix C contains a list of vehicles divided into four classes: self-propelled tracked vehicles, self-propelled wheeled vehicles, construction equipment, and truck-trailer combinations. Each vehicle is identified by its standard nomenclature. The appendix also shows performance categories into which each vehicle falls and the vehicle cone index of each vehicle for 1- and 50-pass performances.

9-31. Performance Categories

Military vehicles are divided into seven arbitrary categories according to the minimum cone index requirements (VCI_1 and VCI_{50}). The range of VCI_1's and VCI_{50}'s for each category and an indication of the type of vehicle that falls under each category (exceptions are numerous) are shown in Table 9-2.

Table 9-2. List of Military Vehicles Showing Vehicle Cone Index (VCI) and Category of Each Vehicle.

Category	Range		Vehicles
	VCI_1	VCI_{50}	
1	12 or less	29 or less	Lightweight vehicles with low contact pressures (less than 2.0 psi).
2	12-21	30-49	Engineer and high-speed tractors with comparatively wide tracks and low contact pressures.
3	21-26	50-59	Tractors with average contact pressures, tanks with comparatively low contact pressures, and some trailed vehicles with very low contact pressures.
4	26-30	60-69	Most medium tanks, tractors with high contact pressures, and all-wheel-drive trucks and trailed vehicles with low contact pressures.
5	31-35	70-79	Most all-wheel-drive trucks, a great number of trailed vehicles, and heavy tanks.
6	35-44	80-99	A great number of all-wheel-drive and rear-wheel-drive trucks, and trailed vehicles intended primarily for highway use.
7	45 or greater	100 or greater	Rear-wheel-drive vehicles and others that generally are not expected to operate off roads, especially in wet soils.

APPENDIX C

CONE INDEX REQUIREMENTS

APPENDIX C

CONE INDEX REQUIREMENTS
Fine-Grained Soils

Tracked Vehicles

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Amphibious Vehicles				
Assault vehicle, full tracked, amphibious M733 FSN 2320-999-4312	10.5	8	20	1
Carrier, cargo, amphibious, tracked: M116	10.9	7	18	1
Landing vehicle, tracked MK4	36.4	19	45	2
Landing vehicle, tracked armored MK4	40.0	20	49	2
Landing vehicle, tracked engineer M1 (LVTE1)	97.5	20	49	2
Landing vehicle, tracked command M5 (LVTP5A1(CMD))	97.5	20	49	2
Landing vehicle, tracked, personnel M5 (LVTP5A1)	87.8	19	45	2
Landing vehicle, tracked howitzer M6 (LVTH6A1)	86.6	19	45	2
Landing vehicle, tracked recovery M1 (LVTR1A1)	82.2	19	44	2
Tank, combat, full tracked: counter-insurgency, amphibious, lt wt, M729 FSN 2350-921-5564	10.5	8	20	1

Notes:

1. Items listed include selected self-propelled vehicles as listed in DA Supply Bulletin 700-20, dated June 1973. Certain items in final development or undergoing field testing have been included where type classification is pending.
2. The vehicle cone index, and category have been calculated from the formulas, curves, and tables in section V of chapter 9. Weight of vehicle was based on normal design loads, equipment, and crews as would be the conditions under full operational deployment in typical off-the-road movements.
3. Amphibious vehicles and engineer tractors have grousers greater than 1-1/2 inches; all other tracked vehicles have grousers less than 1-1/2 inches.

4. One kip equals 1000 pounds (U. S. Customary).

Tracked Vehicles-Continued

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Armored Bulldozers				
Bulldozer, earthmoving M6, tank mtd (tank, combat, 90 mm gun M47)	107.8	25	57	3
Bulldozer, earthmoving M8 (tank, combat, 90mm gun M48)	107.8	22	50	3
Bulldozer, earthmoving tank mtd, M9 (tank, combat, 105mm gun, M60 and M60A1)	116.0	23	53	3
Combat Vehicles				
Armored reconnaissance airborne assault vehicle (General Sheridan) M551	33.6	14	32	2
Flamethrower, self-propelled				
M132	23.3	17	40	2
M132A1	23.9	17	40	2
Gun, antiaircraft artillery, self-propelled: twin 40 mm				
M19A1	41.2	20	48	2
M42	49.5	15	36	2
M42A1	49.5	15	36	2
Gun, field artillery, self-propelled, 155mm, M53, (T97)	96.0	20	47	2
Gun, antitank, self-propelled 90 mm, M56	15.5	10	24	1
Gun, field artillery, self-propelled: 175mm, M107 (T235E1)	62.1	21	50	3
Howitzer, heavy, self-propelled, full tracked 8 inch M55 (T108)	98.0	20	47	2
Howitzer, heavy, self-propelled: 8 inch M110 (T236E1)	58.5	20	47	2
Howitzer, light, self-propelled, full tracked 105mm, M37	46.0	21	49	2

Tracked Vehicles-Continued

Vehicle Description	Vehicle Weight (kips)	VCI$_1$	VCI$_{50}$	Category
Howitzer, light, self-propelled, full tracked: 105mm				
M52	53.0	15	37	2
M52A1	53.0	15	37	2
Howitzer, light, self-propelled: 105mm, M108 (T195E1)	46.9	19	45	2
Howitzer, medium, self-propelled, full tracked 155mm,				
M44	64.0	14	32	2
M44A1	64.0	14	32	2
Howitzer, medium, self-propelled: 155mm, M109 (T196E1)	52.5	25	57	3
Mortar, infantry, self-propelled, full-tracked: 107mm, (4.2 inch) M84	47.1	15	37	2
Rifle, self-propelled, full tracked: multiple, 106mm M50	19.0	11	27	1
Tank, combat, full tracked: 76mm gun,				
M41	51.8	17	40	2
M41A1	51.8	17	40	2
M41A2	51.8	17	40	2
M41A3	51.8	17	40	2
Tank, combat, full tracked: 90mm gun,				
M47	101.8	24	55	3
M48	99.0	20	47	2
M48C	99.0	20	47	2
M48A1	104.0	21	49	2
M48A2	105.0	21	49	2
M48A2C	105.0	21	49	2
M48A3 (M48A1E2)	104.0	21	49	2
Tank, combat, full tracked: 105mm gun,				
M60	102.0	20	48	2
M60A1	102.0	20	48	2

Tracked Vehicles-Continued

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Tank, combat, full tracked: 120mm gun,				
M103	125.0	24	56	3
M103A1	125.0	24	56	3
Tank, combat, full tracked: flamethrower, M67A1	105.8	23	53	3
Vehicle, Combat, engineer, full tracked: 165mm gun, M728 (basic M60A1 tank) FSN 2350-795-1797	115.0	27	62	4
Armored Vehicle Launched Bridges				
Launcher, M48 tank chassis, transporting	96.0	19	45	2
Launcher, M48 tank chassis, transporting, with bridge, armored vehicle launched, scissoring type, class 60: 60-foot	128.0	26	60	4
Launcher, M60A1 chassis, transporting	86.3	15	36	2
Launcher, M60A1 chassis, transporting, with bridge, armored vehicle launched, scissoring type, class 60: 60-foot	115.9	22	51	3
Carriers				
Carrier, cargo, tracked: 6 ton M548	28.0	18	43	2
Carrier, command post, light, tracked:				
M577	23.9	17	40	2
M577A1	24.4	17	40	2
Carrier, command and reconnaissance, armored:				
M114A1	14.7	12	28	1
M114	14.7	12	28	1
Carrier, guided missile equipment, full tracked: M474E2 w/e (PERSHING), FSN 1450-831-6942	11.9	11	26	1
Carrier, 107mm (4.2 in) Mortar, self-propelled:				
M84	47.1	15	37	2
Carrier, personnel, full tracked: armored, M59	42.6	14	34	2

Tracked Vehicles-Continued

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Carrier, personnel, full tracked: armored,				
M113	22.6	16	39	2
M113A1	23.4	17	40	2
Carrier, utility Articulated, M571	8.0	7	19	1
Recovery Vehicles				
Recovery vehicles, full tracked: heavy M51	120.0	21	50	3
Recovery vehicle, full tracked: medium M88	112.0	20	47	2
Recovery vehicle, full tracked: light, armored M578	54.0	21	49	2

Wheeled Vehicles

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Amphibious Vehicles				
Bridge, float, mobile assault, amphibious, (French)	59.4	122	267	7
Lighter, amphibious self-propelled, cargo: 5 ton, LARC-V, design 8005	31.0	30	68	4
Lighter, amphibious self-propelled, cargo: 15 ton LARC-XV, design 8004	75.0	77	170	7
Transporter, amphibious self-propelled, with superstructure, end bay mobile assault bridge (MAB)	60.4	133	291	7
Transporter, amphibious self-propelled, with superstructure, interior bay mobile assault bridge (MAB)	59.4	129	280	7
Missile Vehicles				
Launcher, Rocket: 762mm truck mounted, (Honest John System)				
(M139 chassis), M386, w/e	40.2	29	65	4
(M139D chassis), M289, w/e	47.7	37	82	6
Loader-transporter, guided missile, XM501E2 w/e (HAWK) FSN 1450-768-7046	9.7	32	73	5
Loader-transporter, guided missile, XM501E3 w/e (HAWK) FSN 1450-066-8873	9.5	32	72	5
Transporters				
Transporter, CONEX, 6 x 6, 16 ton	76.0	72	160	7
Trucks				
Truck, cargo: 3/4 ton, 4 x 4,				
M37	7.4	27	61	4
M37B1	7.4	27	61	4

Wheeled Vehicles-Continued

Vehicle Description	Vehicle Weight (kips)	VCI$_1$	VCI$_{50}$	Category
Truck, cargo: 1 1/4 ton, 6 x 6, M561	9.6	19	44	2
Truck, cargo: 2 1/2 ton, 6 x 6, M34	17.2	27	61	4
Truck, cargo: 2 1/2 ton, 6 x 6, M35A1	19.2	26	59	3
Truck, cargo: 2 1/2 ton, 6 x 6				
M36	18.9	27	61	4
M36C	18.9	27	61	4
Truck, cargo: 2 1/2 ton, 6 x 6, M135	18.7	27	61	4
Truck, cargo: 2 1/2 ton, 6 x 6, M211	18.6	26	59	3
Truck, cargo: 5 ton, 6 x 6, M41	29.8	30	67	4
Truck, cargo: 5 ton, 6 x 6, M54	30.6	30	67	4
Truck, cargo: 5 ton, 8 x 8, M656	25.6	18	42	2
Truck, cargo: 8 ton, 4 x 4, M520	43.4	43	97	6
Truck, cargo: 10 ton, 6 x 6,				
M125	49.5	37	84	6
M125A1	49.5	37	84	6
Truck, dump: 2 1/2 ton, 6 x 6, M47	19.2	28	64	4
Truck, dump: 2 1/2 ton, 6 x 6, M59	19.5	27	62	4
Truck, dump: 2 1/2 ton, 6 x 6, M215	18.6	26	59	3
Truck, dump: 2 1/2 ton, 6 x 6, M342A2	20.6	29	65	4
Truck, dump: 5 ton, 6 x 6				
M51	32.7	32	72	5
M51A2	32.7	32	72	5
Truck, maintenance: 3/4 ton, 4 x 4, M201B1	8.8	29	66	4
Truck, maintenance, earthboring: 2 1/2 ton, 6 x 6, V18A/MTQ	19.5	30	67	4

Wheeled Vehicles-Continued

Vehicle Description	Vehicle Weight (kips)	VCI$_1$	VCI$_{50}$	Category
Truck, maintenance, telephone construction and maintenance: 2 1/2 ton 6 x 6, V17A/MTQ	18.8	29	65	4
Truck, platform, utility: 1/2 ton, 4 x 4,				
M274	2.1	14	32	2
M274A1	2.1	14	32	2
M274A2	2.1	14	32	2
M274A3	2.1	14	32	2
M274A4	2.1	14	32	2
Truck, tank, fuel servicing: 2 1/2 ton, 6 x 6, 1200 gal				
M217 (w/600 gal)	19.6	27	62	4
M217C (w/600 gal)	19.6	27	62	4
Truck, tank, fuel servicing: 2500 gal, 4 x 4,				
M559 (GOER) (w/2500 gal)	43.2	49	111	7
(empty)	27.6	24	55	3
Truck, tank, gasoline: 2 1/2 ton, 6 x 6, 1200 gal				
M49 (w/600 gal)	18.8	26	60	4
M49C (w/600 gal)	18.8	26	60	4
Truck, tank, water: 2 1/2 ton, 6 x 6, 1000 gal, M50	18.9	27	61	4
Truck, tank, water: 2 1/2 ton 6 x 6, 1000 gal, M222	18.0	25	57	3
Truck, tractor: 2 1/2 ton 6 x 6 M48 (w/o payload)	11.8	20	46	2
Truck, tractor: 2 1/2 ton 6 x 6, M221 (w/o payload)	11.7	20	46	2
Truck, tractor: 2 1/2 ton 6 x 6, M275 (w/o payload)	11.6	20	46	2
Truck, tractor: 5 ton, 6 x 6,				
M52 (w/o payload)	17.8	21	48	2
M52A1 (w/o payload)	17.8	21	48	2
Truck, tractor: 10 ton 6 x 6,				
M123 (w/o payload)	28.9	21	48	2
M123C (w/o payload)	30.2	22	50	3
M123D (w/o payload)	30.2	22	50	3

Wheeled Vehicles-Continued

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Truck, tractor: 12 ton, 6 x 6, M26A1 (w/o payload)	48.9	35	79	5
Truck, tractor, wrecker: medium, 5 ton, 6 x 6 M246 (w/payload)	44.8	32	73	5
Truck, utility: 1/4 ton 4 x 4, M38	3.6	21	48	2
Truck, utility: 1/4 ton, 4 x 4				
M38A1	3.5	20	47	2
M38A1C	3.5	20	47	2
Truck, utility: 1/4 ton 4 x 4, M151	3.1	19	44	2
Truck, utility: 1/4 ton 4 x 4, lightweight, M422A1	2.6	19	44	2
Truck, van, expansible: 2 1/2 ton, 6 x 6 M292	25.1	34	76	5
Truck, van, shop: 2 1/2 ton, 6 x 6, M109A1	21.0	29	65	4
Truck, van, shop: 2 1/2 ton, 6 x 6, M220	20.4	27	62	4
Truck, wrecker, crane: 2 1/2 ton, 6 x 6, M108	19.8	28	63	4
Truck, wrecker, light: 2 1/2 ton, 6 x 6, M60	24.5	33	74	5
Truck, wrecker, medium: 5 ton, 6 x 6, M62	33.3	33	74	5
Truck, wrecker, medium: 5 ton, 6 x 6, M543	34.4	34	76	5
Truck, wrecker: 10 ton 4 x 4, M553 (GOER)	40.2	42	94	6

Construction Equipment

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Earthmoving Tractors				
Tractor, full tracked, low speed: DED, heavy dbp, w/bulldozer, w/scarifier				
IH Model TD-24-241 FSN 2410-542-2338	54.2	18	43	2
Caterpillar Model D-8 FSN 2410-542-4882	51.0	17	41	2
Tractor, full tracked, low speed: DED, medium dbp, w/bulldozer, cable PCU Caterpillar Model D-7 FSN 2410-277-1280	35.8	16	38	2
Tractor, full tracked, low speed: DED, medium dbp, w/angledozer, w/scarifier, IH Model TD-18-182 FSN 2410-541-7655	39.8	14	34	2
Tractor, full tracked, low speed: DED, medium dbp, w/angledozer, w/scarifier, IH Model TD20-200 FSN 2410-542-2498	40.2	15	36	2
Tractor, full tracked, low speed: DED, light dbp, w/ bulldozer, w/scarifier, Caterpillar Model D-6 FSN 2410-542-4206	16.0	17	40	2
Tractor, full tracked, low speed: DED, light dbp. w/bulldozer, w/scarifier w/wn, Caterpillar Model D-4 FSN 2410-843-6374	17.3	21	49	2
Tractor, full tracked, low speed: DED, light dbp, w/bulldozer, PCU Hydraulic crane; Caterpillar Model 933 FSN 2410-555-1756	16.0	20	48	2
Tractor, full tracked, low speed: DED, medium dbp, w/bulldozer, w/scarifier,				

Construction Equipment-Continued

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Allis Chalmers HD-16M FSN 2410-078-6483	48.5	16	38	2
Caterpillar Model D-7 FSN 2410-782-1130	48.5	16	38	2
Tractor, full tracked, 5 ton universal ballastable, (Universal Engineer Tractor)	36.0	16	39	2
Tractor, wheeled, industrial: DED, medium dbp, w/bulldozer front,				
Clark Model 290M FSN 2420-088-9384	54.2	34	77	5
Caterpillar Model 830M FSN 2420-806-0031	54.2	28	65	4
Cranes and Loaders				
Crane shovel, basic unit, crawler mtd: 2 cu yd 40 ton				
Baldwin-Lima-Hamilton FSN 3810-230-3821	132.6	21	50	3
Crane shovel, basic unit crawler mtd:				
Baldwin-Lima-Hamilton FSN 3810-221-2327	38.2	14	33	2
Crane shovel basic unit, crawler mtd: 10 ton 3/4 cu yd, "UNIT" Model 1020 yd FSN 3810-255-7593	35.0	9	21	1
Crane shovel, crawler w/catwalk: 2 cu yd, 40 ton, Bucyrus-Erie FSN 3810-263-3068	132.6	21	50	3
Crane, revolving, crawler mtd: 30-40 ton,				
Thew Shovel L-82	103.0	22	52	3
Crane, wheel mounted: 3/8 cu yd 5 ton, DED, 4 x 4, rough terrain, air transportable,				
Koehring Model M7 FSN 3810-828-4457	16.0	40	89	6

Construction Equipment-Continued

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}	Category
Crane, wheel mounted 20 ton, 3/4 cu yd, rough terrain, 4 x 4 FSN 3810-060-2735	60.5	42	93	6
Loader, bucket type: full tracked, DED 3 cu yd per min, Haiss Model 77-PC	23.0	20	47	2

STUDENT NOTES

APPENDIX D

SOILS STABILIZATION

1. STABILIZATION WITH PORTLAND CEMENT

Portland cement can be used either to modify and improve the quality of the soil or to transform the soil into a cemented mass, which significantly increases its strength and durability. The amount of cement additive will depend upon whether the soil is to be modified or stabilized. The only limitation to the amount of cement to be used to stabilize or modify a soil pertains to the treatment of the base courses to be used in flexible pavement systems. When a cement-treated base course is to be surfaced with asphaltic concrete, the percent of cement by weight will be limited to 4 percent for Air Force pavements.

a. Cement content for modification of soils.

(1) The amount of cement required to improve the quality of the soil through modification is determined by the trial-and-error approach. If it is desired to reduce the PI of the soil, successive samples of soil-cement mixtures must be prepared at different treatment levels and the PI of each mixture determined.

The minimum cement content that yields the desired PI is selected, but since it was determined based upon the minus 40 fraction of the material, this value must be adjusted to find the design cement content based upon total sample weight expressed as

$$A = 100Bc$$

where

A = design cement content, percent total weight of soil

B = percent passing No. 40 sieve size, expressed as a decimal

c = percent cement required to obtain the desired PI of minus 40 material, expressed as a decimal

(2) If the objective of modification is to improve the gradation of granular soil through the addition of fines, the analysis should be conducted on samples at various treatment levels to determine the minimum acceptable cement content. The determination of cement content to reduce the swell potential of fine-grained plastic soils can be accomplished by molding several samples at various cement contents and soaking the specimens along with untreated specimens for 4 days. The lowest cement content that eliminates the swell potential or reduces the swell characteristics to the minimum becomes the design cement content. The cement content determined to accomplish soil modification should be checked to see whether it provides an unconfined compressive strength great enough to qualify for a reduced thickness design in accordance with criteria established for soil stabilization. (Table 8 and 9)

(3) Cement-modified soil also may be used in frost areas, but in addition to the procedures for mixture design described in (1) and (2) above, cured specimens should be subjected to the 12 freeze-thaw cycles test (but omitting wire-brushing) or other applicable freeze-thaw procedures, followed by frost-susceptibility, determined after freeze-thaw cycling, should meet the requirements set forth for the base course. If cement-modified soil is used as the subgrade,

its frost susceptibility, determined after freeze-thaw cycling, should be used as the basis of the pavement thickness design if the reduced subgrade strength design method is applied.

b. Cement content for cement-stabilized soil. The following procedure is recommended for determining the design cement content for cement-stabilized soils.

(1) Step 1. Determine the classification and gradation of the untreated soil.

The soil must meet the gradation requirments shown in table 2 before it can be used in a reduced thickness design.

(2) Step 2. Select an estimated cement content from table 3 using the soil classification.

(3) Step 3. Using the estimated cement content, determine the compaction curve of the soil-cement mixture.

(4) Step 4. Using the untreated soil gradation characteristics, cement content, and maximum dry density determined in steps 1, 2, and 3, respectively, verify the estimated cement content using table 10 or 11 and figure 5 depending upon soil classification. If the estimated cement content from step 2 varies by more than ±2 percent from the value in table 10 or 11, conduct additional compaction tests, varying the cement content, until the value from table 10 or 11 is within ±2 percent of that used for the moisture-density test.

(5) Step 5. Prepare samples of the soil-cement mixture for unconfined compression and durability tests at the dry density and at the cement content determined in step 4 and at cement contents 2 percent above and 2 percent below that determined in step 4. The samples should be prepared in accordance with TM 5-530 except that when more than 35 percent of the material is retained on the No. 4 sieve, a CBR mold should be used to prepare the specimens. Cure the specimens for 7 days in a humid room before testing. Test three specimens using the unconfined compression test, and subject three specimens to durability tests, either wet-dry tests for pavements located in nonfrost areas or freeze-thaw tests for pavements located in frost areas.

(6) Step 6. Compare the results of the unconfined compressive strength and durability tests with the requirements shown in tables 8 and 9. The lowest cement content, which meets the required unconfined compressive strength requirement and demonstrates the required durability, is the design cement content. If the mixture should meet the durability requirements but not the strength requirements, the mixture is considered to be a modified soil.

Table 8. **Minimum Unconfined Compressive Strengths for Cement, Lime, and Combined Lime-Cement-Fly Ash Stabilized Soils**

Stabilized Soil Layer	Minimum Unconfined Compressive Strength, psi[a]	
	Flexible Pavement	Rigid Pavement
	Air Force and Army	All
Base course	750	500
Subbase course, select material or subgrade	250	200

---

[a] Unconfined compressive strength determined at 7 days for cement stabilization and 28 days for lime or lime-cement-fly ash stabilization.

Table 9. **Durability Requirements**

Type of Soil Stabilized[a]	Maximum Allowable Weight Loss After 12 Wet-Dry or Freeze-Thaw Cycles Percent of Initial Specimen Weight
	Army and Air Force
Granular, PI < 10	11
Granular, PI > 10	8
Silt	8
Clays	6

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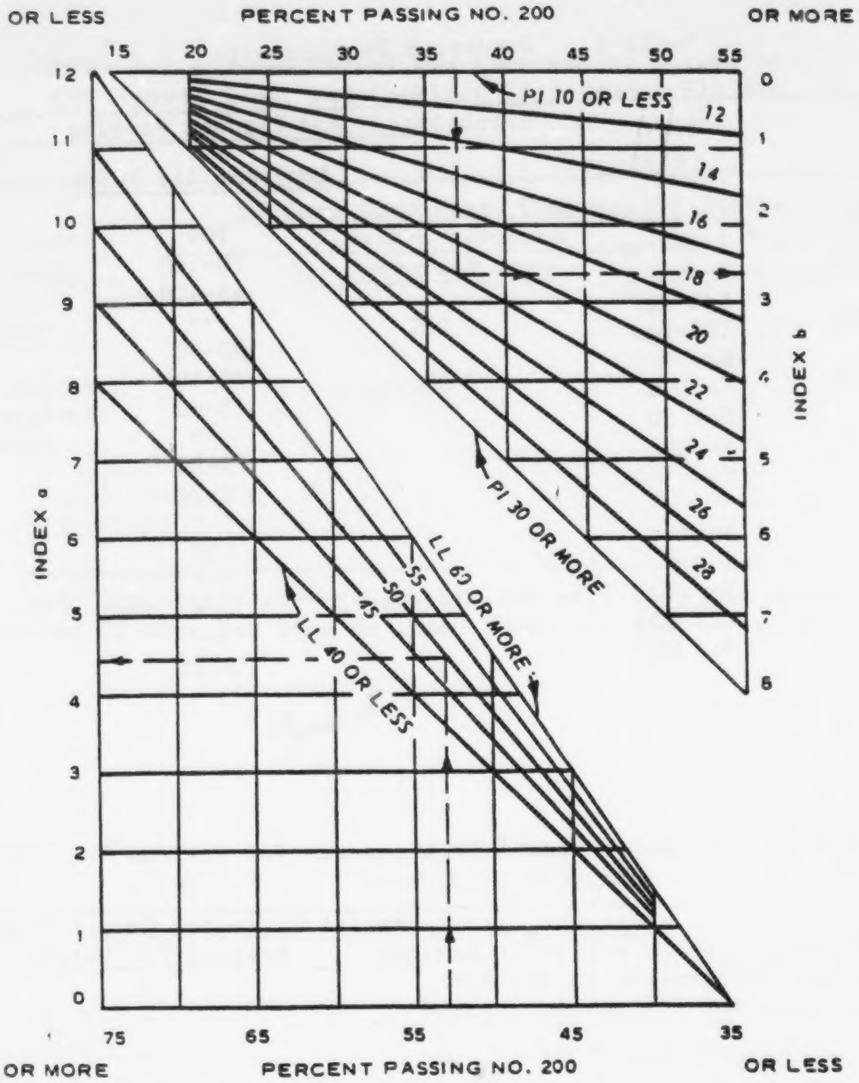
Table 2. Gradation Requirements

Type Course	Sieve Size	Percent Passing
		Army and Air Force
Base	2-in.	100
	1-1/2-in.	70-100
	1-in.	45-100
	3/4-in.	--
	1/2-in.	30-90
	No. 4	20-70
	No. 10	15-60
	No. 30	--
	No. 40	5-40
	No. 200	0-20
Subbase	3-in.	100
	No. 4	--
	No. 10	--
	No. 100	--
	No. 200	0-25

Table 3. Estimated Cement Requirements for Various Soil Types

Soil Classification	Initial Estimated Cement Requirement Percent Dry Weight
GW-SW	5
GP, SW-SM, SW-SC, SW-GM, SW-GC	6
GM, SM, GC, SC, SP-SM, SP-SC, GP-GM, GP-GC, SM-SC, GM-GC	7
SP, CL, ML, ML-CL	10
MH, OH	11
CH	10



NOTE: GROUP INDEX = INDEX a + INDEX b.

(Reprinted from Soil-Cement Laboratory Handbook (EB052.06S) with permission of the Portland Cement Association, Skokie, Illinois.)

Figure 5

Table 10. **Average Cement Requirements for Granular and Sandy Soils**

Material Retained on No. 4 Sieve percent	Material Smaller Than 0.05 mm percent	Cement Content, Percent by Weight Maximum Dry Density, lb/cu ft (Treated Material) 116-120	121-126	127-131	132-137	138-142	143 or more
0-14	0-19	10	9	8	7	6	5
	20-39	9	8	7	7	5	5
	40-50	11	10	9	8	6	5
15-29	0-19	10	9	8	6	5	5
	20-39	9	8	7	6	6	5
	40-50	12	10	9	8	7	6
30-45	0-19	10	8	7	6	5	5
	20-39	11	9	8	7	6	5
	40-50	12	11	10	9	8	6

Note: Base course goes to 70 percent retained on the No. 4 sieve.

Table 11. **Average Cement Requirements for Silty and Clayey Soils**

Group Index[a]	Material Between 0.05 and 0.005 mm percent	Cement Content, Percent by Weight Maximum Dry Density, lb/cu ft (Treated Material) 99-104	105-109	110-115	116-120	121-126	127-131	132 or more
0-3	0-19	12	11	10	8	8	7	7
	20-39	12	11	10	9	8	8	7
	40-59	13	12	11	9	9	8	8
	60 or more	--	--	--	--	--	--	--
3-7	0-19	13	12	11	9	8	7	7
	20-39	13	12	11	10	9	8	8
	40-59	14	13	12	10	10	9	8
	60 or more	15	14	12	11	10	9	9
7-11	0-19	14	13	11	10	9	8	8
	20-39	15	14	11	10	9	9	9
	40-59	16	14	12	11	10	10	9
	60 or more	17	15	13	11	10	10	10
11-15	0-19	15	14	13	12	11	9	9
	20-39	16	15	13	12	11	10	10
	40-59	17	16	14	12	12	11	10
	60 or more	18	16	14	13	12	11	11
15-20	0-19	17	16	14	13	12	11	10
	20-39	18	17	15	14	13	11	11
	40-59	19	18	15	14	14	12	12
	60 or more	20	19	16	15	14	13	12

---

[a] Taken from figure 2.

2 STABILIZATION WITH LIME

Lime can be used either to modify some of the physical properties and thereby improve the quality of a soil or to transform the soil into a stabilized mass, which increases its strength and durability. The amount of lime additive will depend upon whether the soil is to be modified or stabilized. The lime to be used may be either hydrated or quicklime, although the preponderance of stabilization is accomplished using hydrated lime, since quicklime is highly caustic and dangerous to use. The design lime contents determined from the criteria presented herein are for hydrated lime. As a guide, the lime contents determined herein for hydrated lime should be reduced by 25 percent to determine a design content for quicklime.

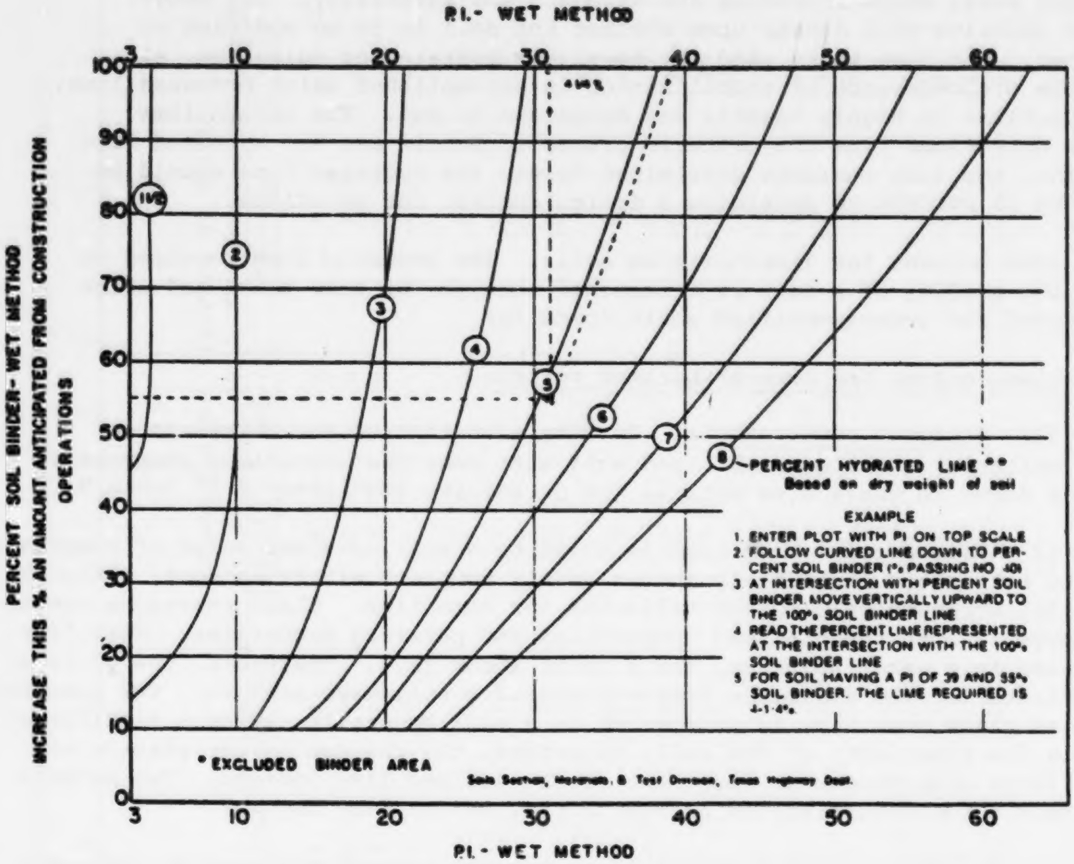
a. Lime content for lime-modified soils. The amount of lime required to improve the quality of a soil is determined through the same trial-and-error process used for cement-modified soils (para 1a).

b. Lime content for lime-stabilized soils.

(1) Strength requirements. To take advantage of the thickness reduction criteria, the lime-stabilized soil must meet the unconfined compressive strengths shown in table 8 as well as the durability requirements in table 9.

(2) Procedures. When lime is added to a soil, a combination of reactions begins to take place immediately and is nearly complete within an hour, although substantial strength gain is not reflected for some time. These reactions result in a change in both the chemical composition and physical properties. Most lime, when placed in a water solution, has a pH of about 12.4. Therefore, the pH is a good indicator of the desirable lime content of a soil-lime mixture. The reaction that takes place when lime is introduced to a soil generally causes a significant change in the plasticity of the soil; therefore, the changes in the plastic and liquid limits also become an indicator of the desired lime content. Two methods are presented for determination of the initial design lime content.

a. Step 1. The preferred method is to prepare several mixtures at different lime treatment levels and determine the the pH of each mixture after 1 hour. The lowest lime content producing the highest pH of the soil-lime mixture is the initial design lime content. Procedures for conducting a pH test on lime-soil mixtures are presented at the end of this section. In frost areas, specimens must be subjected to the freeze-thaw test as discussed in step 2 below. An alternate method of determining an initial design lime content is by the use of figure 2. Specific values required to use figure 2 are the PI and the precent of material passing the No. 40 sieve. These properties are determined from plastic limit and gradation tests on the untreated soil.



EXAMPLE

1. ENTER PLOT WITH PI ON TOP SCALE
2. FOLLOW CURVED LINE DOWN TO PERCENT SOIL BINDER (% PASSING NO. 40)
3. AT INTERSECTION WITH PERCENT SOIL BINDER, MOVE VERTICALLY UPWARD TO THE 100% SOIL BINDER LINE
4. READ THE PERCENT LIME REPRESENTED AT THE INTERSECTION WITH THE 100% SOIL BINDER LINE
5. FOR SOIL HAVING A PI OF 39 AND 55% SOIL BINDER, THE LIME REQUIRED IS 4-1/4%.

Soils Section, Materials & Test Division, Texas Highway Dept.

* Exclude use of chart for materials with less than 10% - No. 40 and cohesionless materials (PI less than 3)

** Percent of relatively pure lime usually 90% or more of Ca and/or Mg hydroxides and 85% or more of which pass the No. 200 sieve. Percentages shown are for stabilizing subgrades and base courses where lasting effects are desired. Satisfactory temporary results are sometimes obtained by the use of as little as 1/2 of above percentages. Reference to cementing strength is implied when such terms as "Lasting Effects" and "Temporary Results" are used.

(Courtesy of the National Lime Association, Washington, D.C.)

Figure 2

b. Step 2. After the initial lime content has been estimated, conduct a compaction test with the lime-soil mixture following the same procedures for soil-cement except that the mixture should be allowed to cure no less than 1 hour and no more than 2 hours in a sealed container before molding. Compaction will be accomplished in five layers using 55 blows of a 10-pound hammer having an 18-inch drop. The moisture-density should be determined at lime contents equal to design +2 and +4 percent for the preferred method and at design ±2 percent for the alternate method. In frost areas, cured specimens should be subjected to the 12 freeze-thaw cycles (but omitting wire brushing) or other applicable freeze-thaw procedures, followed by frost susceptibility determinations in standard laboratory freezing tests. For lime-stabilized or lime-modified soil used in lower layers of the base course, the frost susceptibility, determined after freeze-thaw cycling, should meet the requirements set forth for the base course.

If lime-stabilized or lime-modified soil is used as the subgrade, its frost susceptibility, determined after freeze-thaw cycling, should be used as the basis of the pavement thickness design if the reduced subgrade strength design method is applied.

(c) Step 3. Unconfined compression tests should be performed at the design percent of maximum density on three specimens for each lime content tested. The design value would then be the minimum lime content yielding the required strength. Procedures for the preparation of lime-soil specimens are similar to those used for cement-stabilized soils (para 1b) with two exceptions. After mixing, the lime-soil mixture should be allowed to mellow for not less than 1 hour or more than 2 hours; after compaction, each specimen should be wrapped securely to prevent moisture loss and cured in a constant temperature chamber at 73 degrees ±2 degrees Fahrenheit for 28 days. Procedures for conducting unconfined compression tests are similar to those used for soil-cement specimens except that in lieu of moist curing, the lime-soil specimens should remain securely wrapped until testing.

(d) Step 4. Compare results of the unconfined compressive tests with the criteria in table 8. The design lime content must be the lowest lime content of specimens meeting the strength criteria indicated.

c. Lime and other additives. If lime is used as a preliminary additive to reduce the PI or alter the gradation of a soil preparatory to the addition of the primary stabilizing agent such as bitumen or cement, then the design lime content is the minimum treatment level that will achieve the desired results. For nonplastic and low-PI materials in which lime alone generally is not satisfactory for stabilization, the addition of fly ash may be needed to produce the necessary reaction.

pH TEST TO DETERMINE LIME REQUIREMENTS FOR LIME STABILIZATION

1. Materials. Lime to be used for soil stabilization.

2. Apparatus.

a. pH meter (the pH meter must be equipped with an electrode having a pH range of 14).

b. 150-millilitre (or larger) plastic bottles with screw-top lids.

c. 50-millilitre plastic beakers.

d. Distilled water that is free of CO_2.

e. Balance.

f. Oven.

g. Moisture cans.

3. Procedure.

a. Standarize the pH meter with a buffer solution having a pH of 12.45.

b. Weigh to the nearest 0.01 gram representative samples of air-dried soil, passing the No. 40 sieve and equal to 20.0 grams of oven-dried soil.

c. Pour the soil samples into 150-millilitre plastic bottles with screw-top lids.

d. Add varying percentages of lime, weighed to the nearest 0.01 gram, to the soils. (Lime percentages of 0, 2, 3, 4, 5, 6, 8, and 10, based on the dry soil weight, may be used.)

e. Thoroughly mix soil and dry lime.

f. Add 100 millilitres of distilled water that is CO_2-free to the soil-lime mixtures.

g. Shake the soil-lime and water for a minimum of 30 seconds or until there is no evidence of dry material on the bottom of the bottle.

h. Shake the bottles for 30 seconds every 10 minutes.

i. After 1 hour, transfer part of the slurry to a plastic beaker and measure the pH.

j. Record the pH for each of the soil-lime mixtures. The lowest percent of lime giving a pH of 12.40 is the percent required to stabilize the soil. If the pH does not reach 12.40, the minimum lime content giving the highest pH is required to stabilize the soil.

3. STABILIZATION WITH LIME-CEMENT-FLYASH (LCF)

Stabilization of coarse-grained soils having little or no fines can often be accomplished by the use of the LCF combination. Fly ash is a pozzolanic material, consisting mainly of silicon and aluminum coumpounds that, when mixed with lime and water, forms a hardened cementitious mass capable of obtaining high compressive strengths. Thus, lime and fly ash in combination can often be used successfully in stabilizing granular materials having few fines since the fly ash provides an agent with which the lime can react. In addition to lime and fly ash, a small amount of portland cement is also added to accelerate and increase strength gain. The use of LCF mixtures is not permitted for Air Force Airfield pavements, if the cement content exceeds 4 percent.

a. Suitable materials. Types of materials suitable for the LCF stabilization are coarse-grained soils having no more than 12 percent of the material passing the No. 200 sieve. In addition, the PI of the minus 40 fraction should not exceed 25.

b. Initial mix proportion will be 1% portland cement, 4% lime, 24% fly ash, and 71% soil. Minimum unconfined compressive strength requirements are indicated in table 8. If test specimens do not meet strength requirements, add cement in increments of ½ percent until strengths are adequate. The total quantity of additives should not exceed 15 percent by weight. In frost areas, the LCF mixture should meet the weight loss criteria specified for cement-stabilized soils. The specimens should be compacted in CBR mold.

4. STABILIZATION WITH BITUMEN.

a. Types of bituminous-stabilized soils.

(1) Soil-bitumen. A cohesive soil system made water-resistant with a bituminous admixture.

(2) Sand-bitumen. A system in which sand is cemented together by bituminous material.

(3) Oiled Earth. An earth-road system made resistant to water absorption and abrasion by means of a sprayed application of slow-or medium-curing liquid asphalts.

(4) Bitumen-Waterproofed, Mechanically Stabilized Soil. A system in which two or more soil materials are blended to produce a good gradation of particles from coarse to fine, comparatively small amounts of bitumen are needed, and the processed soil is compacted.

(5) Bitumen-Lime Blend. A system in which small percentages of lime are blended with fine-grained soils to facilitate the penetration and mixing of bitumens into the soil.

b. Soil gradation. The recommended soil gradations for subgrade materials and base or subbase course materials are shown in tables 4 and 5, respectively. Mechanical stabilization may be required to bring soil to proper gradation.

c. Types of bitumen. Bituminous stabilization is generally accomplished using asphalt cement, cutback asphalt, or asphalt emulsions. The type of bitumen to be used depends upon the type of soil to be stabilized, method of construction, and weather conditions. In frost areas, the use of tar as a binder should be avoided because of its high-temperature susceptibility. Asphalts are affected to a less extent by temperature changes, but a grade of asphalt suitable to the prevailing climate should be selected. As a general rule, the most satisfactory results are obtained when the most viscous liquid asphalt that can be readily mixed into the soil is used. For higher quality mixes in which a central plant is used, viscosity-grade asphalt cements should be used. Much bituminous stabilization is performed in place with the bitumen being applied directly on the soil or soil-aggregate system and the mixing and compaction operations being conducted immediately thereafter. For this type of construction, liquid asphalts, i.e., cutbacks and emulsions, are used. Emulsions are preferred over cutbacks because of energy constraints and pollution control efforts. The specific type and grade of bitumen will depend on the characteristics of the aggregate, the type of construction equipment, and climatic conditions. Generally, the following types of bituminous materials will be used for the soil gradation indicated in table 6.

Table 4. Recommended Gradations for Bituminous-Stabilized Subgrade Materials

Sieve Size	Percent Passing
3 in.	100
No. 4	50-100
No. 30	38-100
No. 200	2-30

Table 5. Recommended Gradations for Bituminous-Stabilized Base and Subbase Materials

Sieve Size	1-1/2-in. Maximum	1-in. Maximum	3/4-in. Maximum	1/2-in. Maximum
1-1/2-in.	100	--	--	--
1-in.	84 ± 9	100	--	--
3/4-in.	76 ± 9	83 ± 9	100	--
1/2-in.	66 ± 9	73 ± 9	82 ± 9	100
3/8-in.	59 ± 9	64 ± 9	72 ± 9	83 ± 9
No. 4	45 ± 9	48 ± 9	54 ± 9	62 ± 9
No. 8	35 ± 9	37 ± 9	41 ± 9	47 ± 9
No. 16	27 ± 9	28 ± 9	32 ± 9	36 ± 9
No. 30	20 ± 9	21 ± 9	24 ± 9	28 ± 9
No. 50	14 ± 7	16 ± 7	17 ± 7	20 ± 7
No. 100	9 ± 5	11 ± 5	12 ± 5	14 ± 5
No. 200	5 ± 2	5 ± 2	5 ± 2	5 ± 2

Table 6

(1) Open-graded aggregate.

(a) Rapid- and medium-curing liquid asphalts RC-250, RC-800, and MC-3000.

(b) Medium-setting asphalt emulsion MS-2 and CMS-2.

(2) Well-graded aggregate with little or no material passing the No. 200 sieve.

(a) Rapid- and medium-curing liquid asphalts RC-250, RC-800, MC-250, and MC-800.

(b) Slow-curing liquid asphalts SC-250 and SC-800.

(c) Medium-setting and slow-setting asphalt emulsions MS-2, CMS-2, SS-1, and CSS-1.

(3) Aggregate with a considerable percentage of fine aggregate and material passing the No. 200 sieve.

(a) Medium-curing liquid asphalts MC-250 and MC-800.

(b) Slow-curing liquid asphalts SC-250 and SC-800.

(c) Slow-setting asphalt emulsions SS-1, SS-1h, CSS-1, and CSS-1h.

(d) Medium-setting asphalt emulsions MS-2 and CMS-2.

The simplest type of bituminous stabilization is the application of liquid asphalt to the surface of an unbound aggregate road. For this type of operation, the slow- and medium-curing liquid asphalts SC-70, SC-250, MC-70, and MC-250 are used.

d. Mix Design. Guidance for the design of bituminous-stabilized base and subbase courses in contained in TM 5-822-8, Chapter 9. For subgrade stabilization, the following equation may be used for estimating the preliminary quantity of cutback asphalt to be selected:

Formula #1:

$$p = \frac{0.02(a) + 0.07(b) + 0.15(c) + 0.20(d)}{(100 - S)} \times 100$$

where

p = percent cutback asphalt by weight of dry aggregate

a = percent of mineral aggregate retained on No. 50 sieve

b = percent of mineral aggregate passing No. 50 and retained on No. 100 sieve

c = percent of mineral aggregate passing No. 100 and retained on No. 200 sieve

d = percent of mineral aggregate passing No. 200 sieve

S = percent solvent

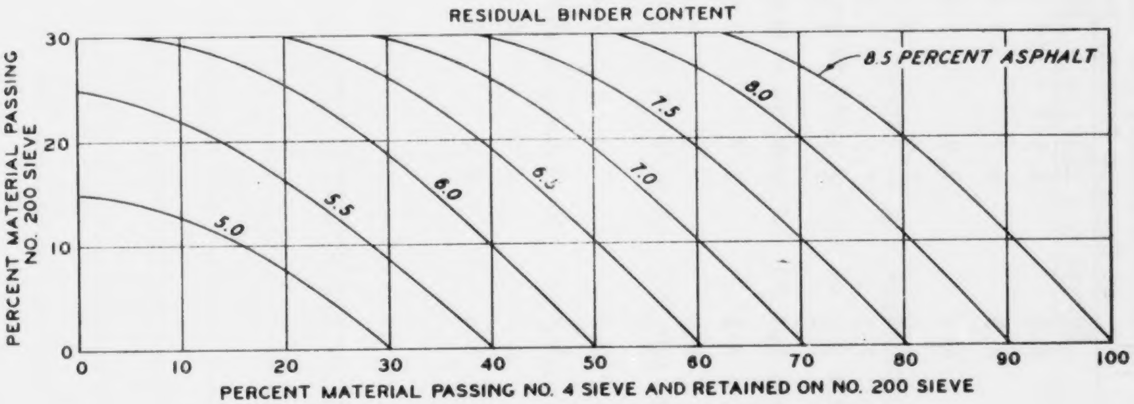
The preliminary quantity of emulsified asphalt to be used in stabilizing subgrades can be determined from table 7. Figure 3a and 3b can be used to find mix design for asphalt cement. The final design content of asphalt should be selected based upon the results of the Marshall Stability test procedure. The minimum Marshall Stability recommended for subgrades if 500 pounds. If a soil does not show increased stability when reasonable amounts of bituminous materials are added, the gradation of the soil should be modified or another type of bituminous material should be used. Poorly graded materials may be improved by the addition of suitable fines containing considerable material passing the No. 200 sieve. The amount of bitumen required for a given soil increases with an increase in percentage of the finer sizes.

Table 7. Emulsified Asphalt Requirements

Percent Passing No. 200 Sieve	Pounds of Emulsified Asphalt per 100 Pounds of Dry Aggregate at Percent Passing No. 10 Sieve					
	≤ 50	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

CLIMATIC ZONES	ASPHALT GRADE, PENETRATION
ARCTIC	100 TO 120
TEMPERATE	85 TO 100
TROPIC	60 TO 70
DESERT	40 TO 50

a. DETERMINATION OF ASPHALT GRADE FOR EXPEDIENT CONSTRUCTION



b. SELECTION OF ASPHALT CEMENT CONTENT

Figure 3. Selection of grade and quantity of asphalt cement

PRACTICAL EXERCISES

1. A site investigation for a proposed airfield produced the soil characteristics of soil A. A possible borrow site had the soil characteristics of soil B. These two soils are to be mechanically stabilized to produce a base course material. Determine the characteristics of the final mix.

Sieve Size	Soil A	Soil B
2"	100	100
1½"	62	75
1"	50	59
3/4"	44	55
3/8"	32	49
#4	21	39
#10	12	23
#40	3	16
#200	1	2
LL	8	16
PL	5	11

SIEVE SIZE	PERCENT PASSING				$\lvert\bar{S}-A\rvert$ (3-1)	$\lvert\bar{S}-B\rvert$ (3-2)	____ A (4)	____ B (5)	PERCENT PASSING, FINAL MIX (4+5)
	SOIL A	SOIL B (1)	SPEC'S S (2)	$\bar{S}$ (3)					
LL					Σ_A	Σ_B			
PL									
PI									

2. The surface course for a proposed road linking two divisions is to be constructed from material from a sand bar and an alluvial terrace. What is the final mix gradation and Atterburg limits?

Sieve Sizes	Sand Bar	Alluvial Terrace
1"	100	100
3/4"	95	80
3/8"	90	51
#4	75	39
#10	63	22
#40	54	13
#200	33	4
LL	40	12
PL	30	8

SIEVE SIZE	PERCENT PASSING				$\lvert\overline{S}-A\rvert$	$\lvert\overline{S}-B\rvert$	____ A	____ B	PERCENT PASSING, FINAL MIX (4+5)
	SOIL A	SOIL B (1)	SPEC'S S (2)	$\overline{S}$ (3)	(3-1)	(3-2)	(4)	(5)	
LL					Σ_A	Σ_B			
PL									
PI									

3. A unit is assigned the mission of quickly providing an expedient road between two organizational units. Only the in-place material can be stabilized. The preliminary site investigation indicates that the soil has a natural strength of 15 CBR, a liquid limit of 30, and a plasticity index of 15 and that 55 percent of the material passes the No. 200 sieve. The soil classifies as a CL material according to the USCS. Only cement, emulsified asphalt, are readily available. Determine type and amount of stabilizer to use and thickness of stabilized layer.

4. A unit is given the mission of providing a road between two organizational task forces. The preliminary site investigation indicates a soil strength of 10 CBR, 80 percent of the material passing the No. 4 sieve, 50 perfent passing the No. 40 sieve, 20 percent passing the No. 200 sieve, and a plasticity index of 12. Only lime and cutback asphalt are available. Determine type and amount of stabilizer to use.

5. A unit is to provide an expedient airfield capable of withstanding 0-V1 aircraft traffic. The preliminary site investigation indicates that the soil classifies as GM according to the USCS and has a natural soil strength of 23 CBR and that drainage conditions are satisfactory. The extended weather forecast indicates that rain can be expected 50 percent of the time for the next 60 days. Determine proper procedures for insuring that this airfield will remain operational.

SOLUTIONS TO HOMEWORK

#1

SIEVE SIZE	PERCENT PASSING				$\lvert\bar{S}-A\rvert$	$\lvert\bar{S}-B\rvert$	0.721 A	0.279 B	PERCENT PASSING, FINAL MIX
	SOIL A	SOIL B (1)	SPEC'S S (2)	$\bar{S}$ (3)	(3-1)	(3-2)	(4)	(5)	(4+5)
1"	100	100	100	100	0	0	72.1	27.9	100.0
3/4"	95	80	85-100	93	2	13	68.5	22.3	90.8
3/8"	90	51	65-100	83	7	32	64.9	14.2	79.1
#4	75	39	55-85	70	5	31	54.1	10.9	65.0
#10	63	22	40-70	55	8	33	45.4	6.1	51.5
#40	54	13	25-45	35	19	22	33.9	3.6	42.5
#200	33	4	10-25	18	15	14	23.8	1.1	24.9
LL	40	12	< 35		$\sum_A = 56$	$\sum_B = 145$	28.8	3.4	32.2
PL	30	8							
PI	10	4	4-9				7.21	1.1	8.3

$\frac{A}{B} = \frac{56}{145}$ ⇒ B = 2.59 A ⇒ 1 part B = 2.59A ⇒ $B = \frac{1}{3.59} = 0.279$ $A = \frac{2.59}{3.59} = 0.721$

#2

SIEVE SIZE	PERCENT PASSING				$\lvert\bar{S}-A\rvert$	$\lvert\bar{S}-B\rvert$	0.278 A	0.722 B	PERCENT PASSING, FINAL MIX
	SOIL A	SOIL B (1)	SPEC'S S (2)	$\bar{S}$ (3)	(3-1)	(3-2)	(4)	(5)	(4+5)
2"	100	100	100	100	0	0	27.8	72.2	100.0
1.5"	62	75	70-100	85	23	10	17.2	54.2	71.4
1"	50	59	55-85	70	20	11	13.9	42.6	56.5
3/4"	44	55	50-80	65	21	10	12.2	39.7	51.9
3/8"	32	49	30-60	45	13	4	8.9	35.4	44.3
#4	21	39	20-50	35	14	4	5.8	28.2	34.0
#10	12	23	15-40	27	15	4	3.3	16.6	19.9
#40	3	16	5-25	15	12	1	0.8	11.6	12.4
#200	1	2	0-10	5	4	3	0.3	1.4	1.7
LL	8	16	<25		$\sum_A = 122$	$\sum_B = 47$	2.2	11.6	13.8
PL	5	11							
PI	3	5	<5				0.8	3.6	4.4

$\frac{A}{B} = \frac{122}{47} = 2.60$ ⇒ A = 2.60 B ⇒ 1 part A = 2.6 part B ⇒ $A = \frac{1}{3.6} = 0.278$

$B = \frac{2.6}{3.6} = 0.722$

#3. CBR = 15
LL = 30
PI = 15
USCS: CL
55% passes #200

Entering figure 1 with % fines, the point plots somewhere in Area 3. Table 1 denotes that only lime and portland cement can be used. Only lime is available and PI is greater than 12. Table 3 indicates to use 10% portland cement: Entering figure 4 at CBR of 15 and intersecting curve #4 gives stabilization of 9".

#4. CBR = 10
PI = 12
80% passes #4
50% passes #40
20% pases #200

Percent that passes #4 and retained on #200 is 80-20 = 60%. Entering Figure 1 with 60% and 20%, plots in Area 1C. Table 1 denotes four materials to use. However, only lime and cutback asphalt are available and PI is greater than 10 so lime is used. Entering figure 2 with PI of 12, and 50% passing #4 gives lime content of 12%.

#5.

From a study of the design curve for O-V1 aircraft (Figure 4), it can be seen that a 23-CBR soil strength is adequate. However, a reduction in strength can be anticipated due to the predicted rainfall.

A serviceable airfield can be constructed on this site and expected to remain operable during the predicted rainfall if (a) the site is shaped to the proper crown and grade, (b) an adequate drainage system is provided, and (c) the surface of the airfield is waterproofed.

Because of the aggregate or coarse-grained natural soil, a single or multiple asphalt surface treatment should be applied to waterproof the airfield.

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